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Draughtsmen.

Design of Structural Welded Details.

By F. H. ABRAHAMS, A.M.I.Struct.E., M.Inst.W.

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INTRODUCTION.

In changing over from riveted to welded construction, it should not be assumed that all that is required is to maintain the same general type of construction with rivets replaced by welds of equal strength. An entirely different technique is called for in welded design. For example, the end connection of a beam is usually made with web cleats in riveted work but the cleats may be dispensed with altogether in welded work. Similarly with a plate girder, flange angles are required in riveted construction to connect the web plate to the flange plates, but in welded construction the flange angles may be omitted and the web plate welded directly to the flange plates. A much better understanding of the requirements for welded work will be obtained by assuming that the welded construction is an alternative to a steel casting rather than to a built up riveted construction. The draughtsman is, in effect, building up the equivalent of a casting by introducing weld metal in place of the fillets in a casting.

Rolled steel sections were of course designed to meet the requirements for riveted construction, and it is not surprising that they are often unsuitable and uneconomical for welded work. When this is the case, the designer must either make the best possible use of such sections as come nearest to his requirements, or alternatively build up his own sections from plates or flats welded together. The latter method is often the more economical in spite of the amount of welding involved. For instance, a riveted stanchion shaft may consist of a rolled joist with flange plates, but in welded work it may be more economical to build up an H section from plates and flats, varying the width and thickness of the flanges at different heights to suit the load.

It will therefore be found that welding details shew considerable simplification and are more economical in material compared with riveted work, as sections can be changed more easily to suit load variations and joint details may consist solely of weld metal without added joint material. This is a most important point to bear in mind in assessing the relative costs of welded and riveted designs, as although the actual operation of making a welded connection may be more expensive than that of drilling and riveting (though not always so), there is often considerable saving of material, and consequently of the labour employed in preparing the material thus saved. This does not necessarily mean that the overall cost of a welded connection will be cheaper than that of a riveted connection, but it does mean that where material can be saved and details simplified a case has been established for examining relative costs.

Furthermore, with certain constructions—as every draughtsman will have discovered in the course of his experience—it is very difficult to produce a riveted or bolted joint which is wholly satisfactory and at the same time reasonably simple, whereas the same joint if welded can be comparatively simple and occasion much less trouble. In such cases the welded construction is preferable, as simplicity of detail is always desirable, and is indeed a first principle in sound design.

To some extent, calculations given in this pamphlet are approximate, erring slightly on the strong side and making suitably safe assumptions. This seems desirable until welding technique and practice are more generally understood, and especially so for the draughtsman who is feeling his way towards a proper grasp of the subject.

It will be appreciated that in a work of this size it is not possible to deal with problems of fabrication in more than general terms. Many of these problems can, in fact, only be solved by the method of trial and error, and in welding work more than in most forms of construction experience is the best teacher. Useful information can however be obtained on the subject from such works as the "Report on the Treatment of Welded Structures" issued by the Institution of Structural Engineers—now somewhat out of date but still useful as an introductory guide—and also from the monthly Journal of the Institute of Welding, which usually gives references for further reading.

In the calculations which follow it has been assumed that the reader possesses a knowledge of elementary theory of structures, and attention has been concentrated on applying standard design procedure to the particular problem of welded details. Most of the calculations are, however, of a simple nature, and should be followed without much difficulty.

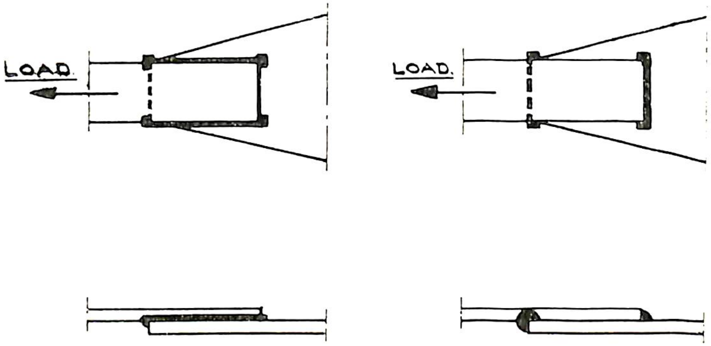
DESIGN OF STRUCTURAL WELDED DETAILS.

F. H. Abrahams, A.M.I.Struct.E., M.Inst.W.

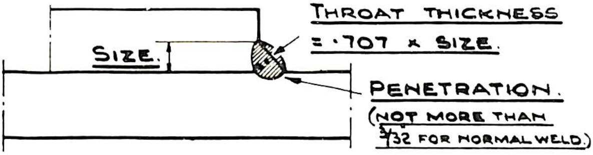
PART I.

GENERAL NOTES ON WELDING.

Fig. 1 shews typical examples of side and end fillet welds. The basis of weld strength is the thickness of the weld at the throat, and the effective area of a fillet weld for design purposes is the throat thickness multiplied by the effective length. The strength



TYPICAL SIDE FILLET. TYPICAL END FILLET.


SIZE.
THROAT THICKNESS = ·707 x SIZE.
PENETRATION.
(NOT MORE THAN 1/32 FOR NORMAL WELD.)

ENLARGED DETAIL OF SIDE & END FILLETS.

Fig. 1.

of a weld can most conveniently be stated as so many tons per lineal inch run of weld ; this is the usual procedure in design and the method generally adopted in tables of strengths of welds. As weld strength is based on calculated throat thickness, it will be obvious that if too much weld metal is deposited and the shape of the weld is too convex the connection is wasteful, while if too little metal is deposited and the shape is too concave the connection will be too weak. The correct profile should leave the weld slightly convex, as shewn in the enlarged detail, and the length of run which can be obtained from a standard electrode is based on such a profile.

The short returns to the welds in Fig. 1 are not included in strength calculations, but are necessary to ensure that the full length of the weld run is effective. This is due to the fact that a small crater is usually formed at the beginning and end of a run, and it is unsafe to assume that such craters are effective in taking load. If it is not possible to include a return, the effective design length should be assumed to be the actual length less twice the weld size. Thus a $\frac{1}{4}''$ fillet $6''$ long would be assumed to have an effective length of $5\frac{1}{2}''$.

Until the publication of B.S. 449, 1948, covering the use of structural steel in buildings, permissible stresses in fillet welds

TABLE I.

Strength in tons per Lineal Inch of Fillet Welds.

(L.C.C. Regulations, *December*, 1937).

Size of Fillet	Throat Thickness	End Fillets at 6 tons/sq. in.	Side Fillets at 5 tons/sq. in.
$\frac{1}{8}''$	0·088″	0·53	0·44
$\frac{3}{16}''$	0·132″	0·79	0·66
$\frac{1}{4}''$	0·177″	1·06	0·89
$\frac{5}{16}''$	0·221″	1·33	1·11
$\frac{3}{8}''$	0·265″	1·59	1·33
$\frac{1}{2}''$	0·354″	2·12	1·77
$\frac{5}{8}''$	0·442″	2·65	2·21
$\frac{3}{4}''$	0·530″	3·18	2·65
$\frac{7}{8}''$	0·619″	3·71	3·10
1″	0·707″	4·24	3·54

were usually taken from L.C.C. Regulations, dated December, 1937, which specify a permissible stress of 6 tons per square inch for end fillets and 5 tons per square inch for side fillets. These stresses are still sometimes specified, on the principle of differentiating between side and end fillets, and Table 1 gives the strength per lineal inch of the usual range of fillet welds in accordance with these regulations.

B.S. 449, 1948—Clause 24a (ii)—specifies that the permissible stress in fillet welds, calculated on an area equal to the effective length of the weld multiplied by the throat thickness, shall not exceed the permissible shear stress of the parent metal. For mild steel manufactured in accordance with B.S. 15, 1948, the maximum allowable shear stress is 6½ tons per square inch. This allowable stress holds good for all unstiffened webs where the ratio of thickness to depth does not exceed one-seventy-fifth (usually rolled joists or channels), but for stiffened webs (usually plate girders) this allowable stress may have to be reduced in accordance with Clause 20b. Generally speaking B.S. 449, 1948, appears to be an advance on the earlier L.C.C. Regulations, and should be followed in cases where the designer has free choice. In cases where the structure generally is to be designed to B.S. 449, these stresses are of course obligatory. Table 2 gives the strength of fillet

TABLE 2.

Strength in tons per Lineal Inch of Side & End Fillet Welds.

(For various allowable stresses in accordance with B.S. 449, 1948).

Size of Fillet	Throat Thickness	Allowable Stress in tons per sq. in.							
		6·5	6·0	5·5	5·0	4·5	4·0	3·5	3·0
$\frac{1}{8}$″	0·088″	0·57	0·53	0·48	0·44	0·40	0·35	0·31	0·26
$\frac{3}{16}$″	0·132″	0·86	0·79	0·73	0·66	0·59	0·53	0·46	0·40
$\frac{1}{4}$″	0·177″	1·15	1·06	0·97	0·89	0·80	0·71	0·62	0·53
$\frac{5}{16}$″	0·221″	1·44	1·33	1·22	1·11	0·99	0·88	0·77	0·66
$\frac{3}{8}$″	0·265″	1·72	1·59	1·46	1·33	1·19	1·06	0·93	0·80
$\frac{1}{2}$″	0·354″	2·30	2·12	1·95	1/77	1·59	1·42	1·24	1·06
$\frac{5}{8}$″	0·442″	2·87	2·65	2·43	2·21	1·99	1·77	1·55	1·33
$\frac{3}{4}$″	0·530″	3·45	3·18	2·92	2·65	2·39	2·12	1·86	1·59
$\frac{7}{8}$″	0·619″	4·02	3·71	3·40	3·10	2·79	2·48	2·17	1·86
1″	0·707″	4.60	4·24	3·89	3·54	3·18	2·83	2·47	2·12

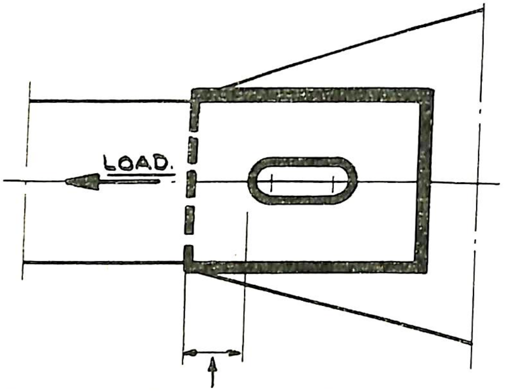
Values for intermediate stresses may be found by interpolation.

welds per lineal inch for various allowable stresses up to a maximum of 6½ tons per square inch, and these stresses apply equally to both side fillets and end fillets.*

In the case of fillet welds subject to combined bending and direct load, examples of which will be found in following pages, B.S. 449—Clause 24a (iii)—specifies that the maximum resultant stress, calculated as the vector sum, shall not exceed the above mentioned allowable stress.

With regard to butt welds, the L.C.C. Regulations already referred to allow a permissible stress of 8 tons per square inch in tension and compression. B.S. 449—Clause 24a (i)—states that the permissible stress in butt welds shall not exceed the permissible tensile compressive and shear stresses in the parent metal, and this appears to be a much more satisfactory control. This means that in the case of a complete penetration butt weld the two parts may be assumed to be effectively joined together, and no calculations are therefore required for the weld. In the case of incomplete penetration butt welds, welded from one side only, the assumed throat thickness for design should be in accordance with Clause 44h, B.S. 449.

Fig. 2 indicates a method of increasing the length of fillet weld in a connection, where loads are heavy, by the introduction of a



WELD METAL IN THIS SPACE
MUST REPLACE SLOT AREA.

TYPICAL EXAMPLE OF SLOT.

Fig. 2.

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*For special structures such as cranes, British Standards specify lower allowable stresses than those given in B.S. 449. In all such cases reference should be made to the appropriate Standard.

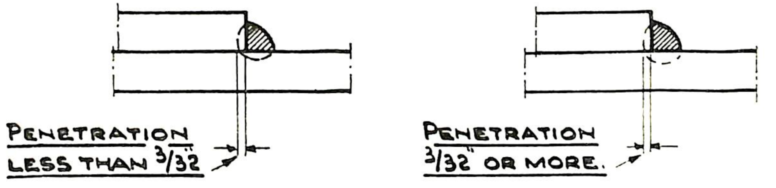
slot. The method is not to be recommended and should only be used where unavoidable, as slot welds are somewhat difficult to lay satisfactorily and there is also the expense of making the slot. Where this method is used it is important to ensure that the weld metal does not fill the slot, or the effect of the procedure is partly nullified. This means that the slot must be of sufficient width to enable the weld metal to be deposited round the perimeter without difficulty. Care must also be taken to ensure that there is a sufficient length of weld metal in advance of the slot to transmit load equivalent to the sectional area of the slot. For example, if the slot reduces the sectional area of the bar by 20%, then at least 20% of the weld metal must be in advance of the slot to transfer that percentage of the load before the bar is weakened by loss of metal. This principle is exactly the same as for designing rivets in advance.

In all cases where slots are employed, the construction of the slot must be in accordance with Clause 45k of B.S. 449.

For all end connections made with fillet welds, whether with or without slots, a single end fillet should never be used. Either two end fillets should be included, or—if that is not possible—two side fillets with one end fillet, or two side fillets without end fillets.

Deep Penetration Fillet Welds.

All references to fillet welds up to this stage, together with allowable loads, are to welds with normal penetration (not exceeding $\frac{3}{32}$") as shewn in Fig. 1. It sometimes happens that it is necessary to increase weld values and it is not possible or is undesirable to increase the normal weld size. In such cases welds may be employed with deeper penetration, as shewn in Fig. 3, in which case the assumed weld size for design may be taken as the



(a) NORMAL FILLET WELD. (b) DEEP PENETRATION FILLET WELD.

DEEP PENETRATION WELD.

Fig. 3.

nominal leg length plus the penetration beyond the root.* Welds of this type should never be used except where unavoidable, as there will always be some doubt that the increased penetration has been maintained for the full length of the weld, and where they are used, evidence must be produced to prove the depth of the penetration.

Obtuse Angle Fillet Welds.

In cases where the angle between the parts exceeds 90° but does not exceed 120°, the assumed throat thickness is to be less than for a normal square angle fillet. For angles of more than 90° but not exceeding 100°, the throat thickness is assumed to be the fillet size multiplied by 0·65 instead of by the standard 0·707 in the case of a square angle. For angles over 100° but not exceeding 106° the multiplier is to be 0·60. For angles over 106° but not exceeding 113° the multiplier is to be 0·55, and for angles over 113° but not exceeding 120° the multiplier is to be 0·50. Angles of more than 120° between the fusion faces may not be used unless proved by actual tests to develop the required strength.

Acute Angle Fillet Welds.

Unless proved by tests to develop the required strength, the angle between fusion faces must not be less than 60° for flat or downhand welding, not less than 70° for vertical or horizontal-vertical welding, and not less than 80° for overhead welding. In each of these cases the throat thickness may be assumed to be 0·707 times the fillet size as for a square angle.

Minimum Sizes of Fillet Welds (B.S. 1856, 1952—Table 1).

For single run fillet welds, or for the first run of multi-run fillets, where the parts are $\frac{3}{8}''$ or more in thickness, the minimum allowable size of fillet is to be $\frac{3}{16}''$ where the thicker part is $\frac{3}{8}''$ up to $\frac{3}{4}''$ thick, $\frac{1}{4}''$ where the thicker part is more than $\frac{3}{4}''$ up to $1\frac{1}{4}''$ thick, and $\frac{5}{16}''$ where the thicker part is over $1\frac{1}{4}''$ thick. Where the thicker part is more than 2″ thick, or where the thinner part has a thickness less than the minimum weld size, special precautions must be taken under proper supervision to ensure that the welds are sound.

Tee Fillets (Fig. 4).

L.C.C. Regulations of December, 1937, specify that where the welds of a tee-fillet are in tension they should be designed as side fillets at 5 tons per square inch, but where they are in compression they may be designed as end fillets at 6 tons per square inch. When designing to B.S. 449, the differentiation between fillets in tension

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*See B.S. 1856, 1952—Clause 18.

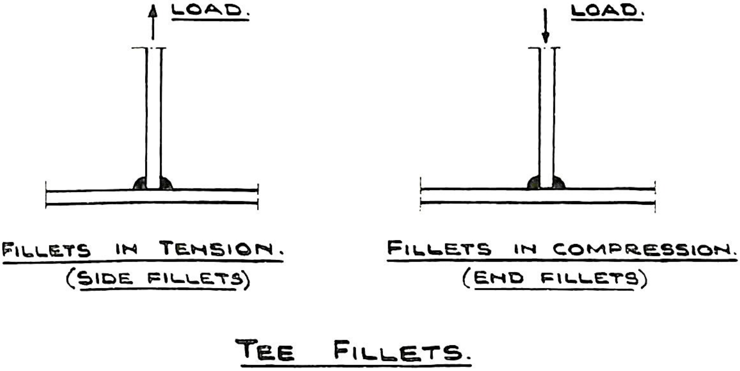


Fig. 4.

and compression may be ignored, as in either case the allowable stress is to be the same as the allowable shear stress in the parent metal.

Diagonal Fillets (Fig. 5).

These are obviously neither side nor end fillets, but L.C.C. Regulations specify that the allowable working stress should be 5 tons per square inch as for side fillets, and when designing to these Regulations this figure must be used. For designs to B.S. 449,

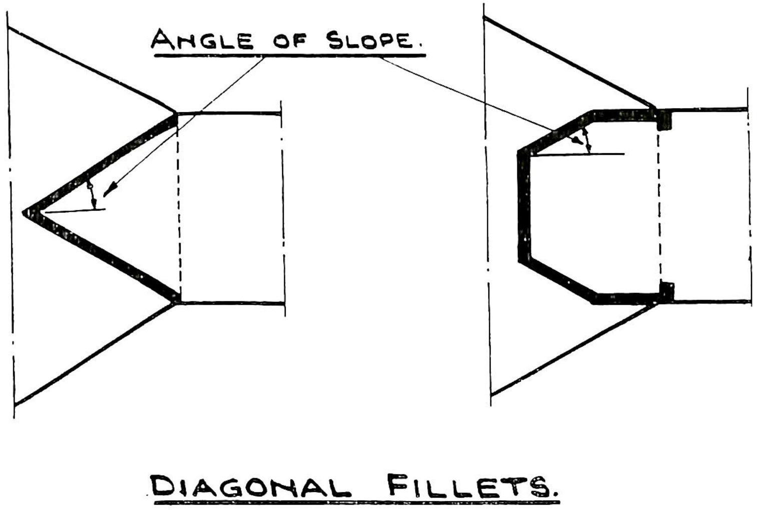
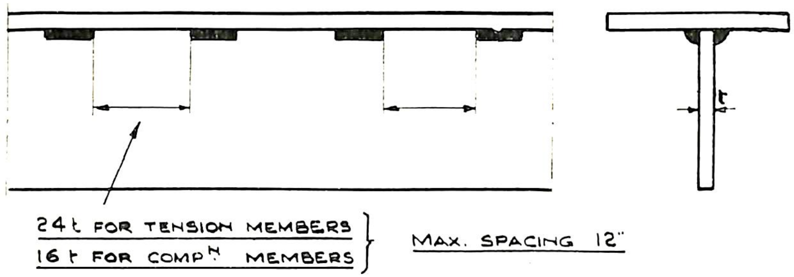


Fig. 5.

the allowable working stress will be the allowable shear stress in the parent metal.

Intermittent Fillet Welds (Fig. 6).

Where continuous welds are not necessary for strength, or for weathering or other similar purpose, intermittent welding may be adopted. Apart from the saving in cost where the welds are intermittent instead of continuous, a further most important point is that with intermittent welds the distortion of the parent metal is less than with continuous welds, and any procedure which minimises this most troublesome problem of distortion is to be welcomed. (Further notes on distortion will be found on later pages). For tension members the gap between the welds is not to exceed 24 times the thickness of the thinner plate, and for compression members 16 times the thickness of the thinner plate, but in no case is the gap to be more than 12″ when welds are subject to stress.



INTERMITTENT FILLET WELDS.

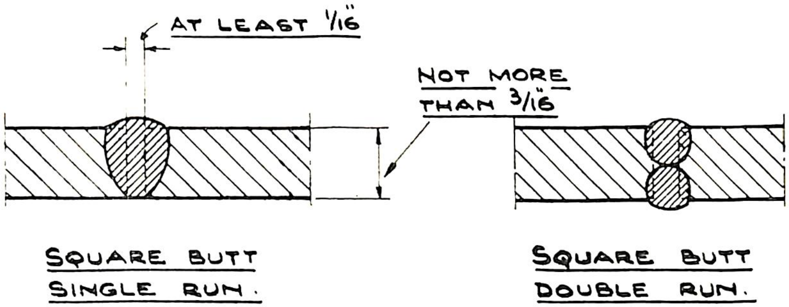
Fig. 6.

In cases where intermittent welds are not subject to calculated stress, but serve solely to hold the component parts of a built up member in proper alignment, the pitch of the effective lengths of welds—centre to centre—in tension members may be widened to 3′ 6″ on line, whether the welds on the two sides are opposite each other or staggered. For built up compression members where the intermittent welds are not subject to calculated stress the pitch of the welds is to be such that the slenderness ratio of the component parts is not to be more than 40 or 0·6 of the slenderness ratio of the member as a whole, taken about the most unfavourable axis.

In all cases where intermittent welds under stress are used for loaded members the welds on both sides must extend to the end of the member, though this requirement need not apply to subsidiary fittings such as stiffeners.

Square Butt Welds (Fig. 7).

L.C.C. Regulations specify a maximum plate thickness of $\frac{3}{16}$" for these welds, with a gap of not less than $\frac{1}{16}$", although quite satisfactory square butts can be made with plates up to at least $\frac{3}{8}$" thick. The objection to a square butt for plates over $\frac{3}{16}$" is the tendency of the weld metal as it cools and contracts to draw the plates together as the welding advances thus reducing the gap and decreasing the effective penetration. This can be obviated by introducing tack welds at intervals to hold the plates in correct position and to maintain the gap.



SQUARE BUTT SINGLE RUN.

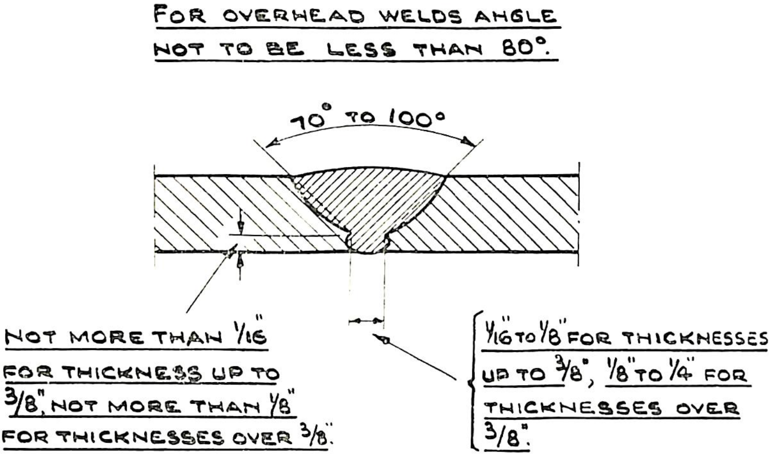
SQUARE BUTT DOUBLE RUN.

SQUARE BUTT WELDS TO L.C.C. REGULATIONS.

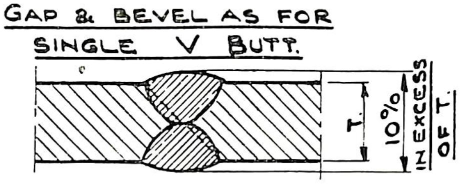
Fig. 7.

B.S. 449 does not specify maximum sizes, but refers to B.S. 538 "Metal Arc Welding as Applied to Steel Structures" for these particulars. B.S. 538 fixes a maximum plate thickness of $\frac{1}{2}$" for square butt welds, but states that they shall not be used in any case with overhead welding. For plates up to $\frac{3}{16}$" thick, B.S. 538 states that the joint must be made by welding from both sides of the plate, and for plates from $\frac{3}{16}$" to $\frac{1}{2}$" thick a mild steel backing strip is to be used for horizontal welding, while with vertical welding the backing strip may be included or omitted as desired. With all square butts over $\frac{3}{16}$", B.S. 538 specifies that tests shall be carried out. Fig. 7 gives particulars of square butt welds to L.C.C. Regulations. For particulars of square butts to B.S. 538 for various plate thicknesses and conditions reference should be made to the diagrams in that specification.

Additional information on Square Butt welds is given in B.S. 1856, 1952, Appendix A, to which reference should be made. This specification states that Appendix A is not mandatory, but the


FOR OVERHEAD WELDS ANGLE NOT TO BE LESS THAN 80°.
70° TO 100°
NOT MORE THAN 1/16" FOR THICKNESS UP TO 3/8", NOT MORE THAN 1/8" FOR THICKNESSES OVER 3/8".
1/16" TO 1/8" FOR THICKNESSES UP TO 3/8", 1/8" TO 1/4" FOR THICKNESSES OVER 3/8".

SINGLE V BUTT.



DOUBLE V BUTT.

Fig. 8.

information given is necessarily based upon more up-to-date practice than B.S. 538 and except when designs are specifically required to be in accordance with B.S. 449 (which refers to B.S. 538 for details of construction) it seems desirable to give preference to the suggestions offered in this Appendix rather than to adhere rigidly to B.S. 538.

In this connection it might be noted that early issues of B.S. Specifications dealing with welding are all likely to be revised or withdrawn in due course, and whenever there appears to be any conflict between these Specifications and those of later issue, preference should be given to the latter in cases where free choice is possible.

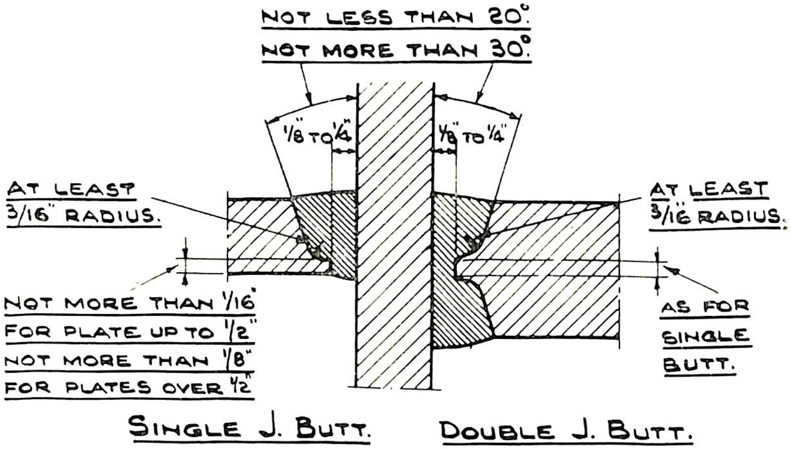
V-Butt Welds (Fig. 8).

For plates over $\frac{3}{16}''$ thick, single or double V-butt welds are most generally used for structural work. Double V-butts are used for plates over $\frac{5}{8}''$ thick for economy in champering and to reduce the width of the gap. They may also be used for plates thinner than $\frac{5}{8}''$ if it is not essential to grind the weld to give a flush surface for bearing purposes. Generally however for plates from $\frac{3}{16}''$ to $\frac{5}{8}''$ thick single V-butts are mostly used, together with a sealing run on the far side to ensure that the full sectional area of weld is obtained. The sealing run can, if desired, be dressed flush with the plate to give a level bearing. For general structural purposes, V-butts—either single or double—cover all normal requirements for butt welding, and should be regarded as the standard procedure.

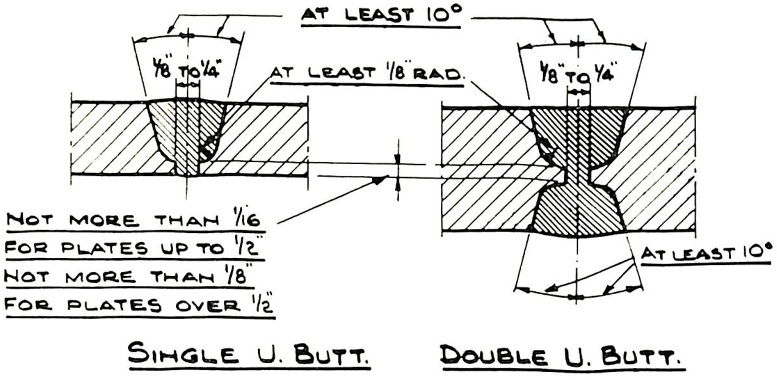
For downhand welding, V-butts are the easiest of all welds to lay, as they are so shaped that the molten metal assumes the desired profile by gravity. They are also comparatively cheap, as maximum welding speeds can be obtained with this type of weld. When for any reason it is not possible to include the sealing run in a single V-butt, specification requires that the throat thickness shall be not less than seven-eighths of the plate thickness and requires evidence to be produced to shew that this requirement has been met. For stress calculation in such cases, to allow for the eccentricity of the weld metal relative to the parent metal, a nominal throat thickness not exceeding five-eighths of the plate thickness is to be assumed. In any case unsealed butt welds of any type are not to be used when the plate is subject to a bending moment about the longitudinal axis of the weld. Neither are such welds to be used when the load includes impact or vibration effects.

J-Butt and U-Butt Welds (Fig. 9).

These welds are rarely used in structural practice, and only when special circumstances make it impossible to employ a standard



J. BUTT WELDS.



J. & U. BUTT WELDS.

Fig. 9.

V-butt. The labour in preparation is much greater than for a V-butt with no corresponding advantages for normal work. J-butts or U-butts become necessary when it is not possible for the operator to see clearly the work in progress had a V-butt been specified, or where a V-butt does not allow proper access for the electrode to the surfaces to be welded, but it is not often in normal structural work that this will occur. Rather than use such welds it is much better to revise the detail to suit a standard V-butt whenever possible. Requirements for incomplete penetration, that is, without sealing run, apply equally to these welds as for V-butt welds.

Where two plates of unequal thicknesses have to be butt-welded together, it is obvious that at the joint the thinner plate is strong enough to take the load, and it is therefore necessary for the weld to develop the strength of the thinner plate only. To obtain a satisfactory joint the thicker plate should be bevelled down to the thickness of the thinner plate with a slope of 1 in 5, as shown in Fig. 10. When for any reason it is not desired to bevel the thicker plate, the weld metal should be built up at the junction with the thicker plate to a dimension of at least 25% greater than that of the thinner plate, or alternatively to the thickness of the thicker plate, whichever is the lesser.

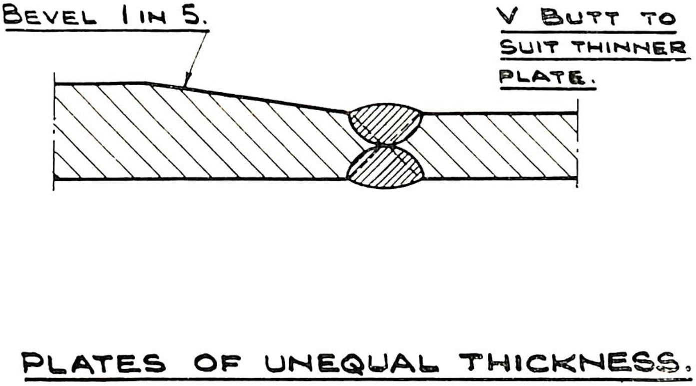


Fig. 10.

Welds Between Horizontal and Vertical Plates.

Good quality fillet welds should have a straight, even leg length with a regular smooth surface and slightly convex profile, together with the correct penetration over the complete run. The quality of the weld is dependent upon the welding current, and upon the size, operational angle, and speed of travel of the electrode. Incorrect combinations of all these factors will almost certainly produce faults which in some cases can only be remedied effectively by cutting out and re-welding. The main faults are :—

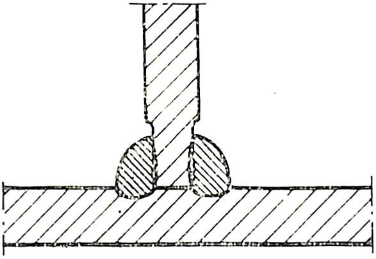
(1) Incorrect weld contour due to weld metal flattening out on the horizontal plate, caused by the electrode being held at the wrong angle.

(2) Undercutting of the vertical plate, due to the electrode being too large, and/or the travel speed being too low or irregular, thus creating a definite point of weakness at a place where stress concentration occurs. (See Fig. 11).

(3) Inadequate penetration and a lumpy bead surface due to the current being too low.

(4) Rough burnt surfaces of plate with weld spatter due to the current being too high.

In trying to avoid any one of these faults it is very easy for the operator to fall into one or more of the others, and when taking out electrode quantities it is advisable to assume that more weld metal will be deposited than is theoretically necessary. In cases where undercutting has occurred the loss in area must be made good with an additional run if the weld is not cut out and relaid.



UNDERCUTTING OF VERTICAL PLATE.

Fig. 11.

Vertical Welding.

Vertical welds may be laid either downward or upward, but preferably upward. The heat absorbing property of the plates freezes the weld metal solid before it has had time to fall providing the arc is not concentrated too long in any one locality. Surface tension also resists the tendency of the molten metal to fall. In downward welds only light runs should be attempted or the molten metal will fall by gravity before it has had time to freeze, and any attempt to make a larger run only results in melting away metal already deposited. Downward welding is very suitable for sealing runs, and may also be used for seams up to ¼″. Upward welding should always be used for heavier seams so that the deposited metal when freezing provides support for succeeding runs.

Fig. 12 illustrates a suitable and usually satisfactory method of laying vertical welds, both for single pass and multi-pass welding. The first run is laid in an upward direction from point 1 to point 2, and the second run also upward, from point 3 to point 4, the gap between points 1 and 4 being not greater than can be filled in with a single electrode. This gap is now filled in, also in an upward direction. If the weld is of the multi-pass type, the second and following passes are laid in the same manner as the first pass, all these passes being convex in shape. The final pass, intended to give a good finish to the weld, is made downwards with a concave surface as smooth as possible.

Typical faults in vertical welding are :—

(1) Undercutting due to the current being too high, the travel speed too low, or the electrode too large.

(2) Slag cavities due to the current being too high, or the travel speed too high.

Generally, with vertical welding—whether upward or downward—the first run should be made with a small electrode of about 10 gauge to ensure good penetration.

Overhead Welding.

This is a somewhat difficult operation, owing to the tendency of the molten metal to fall by gravity, and should only be attempted when no other procedure is possible. The globules of metal passing across the arc must be frozen in the shortest possible space of time, and special "overhead" electrodes possessing the necessary properties for quick freezing are usually employed.

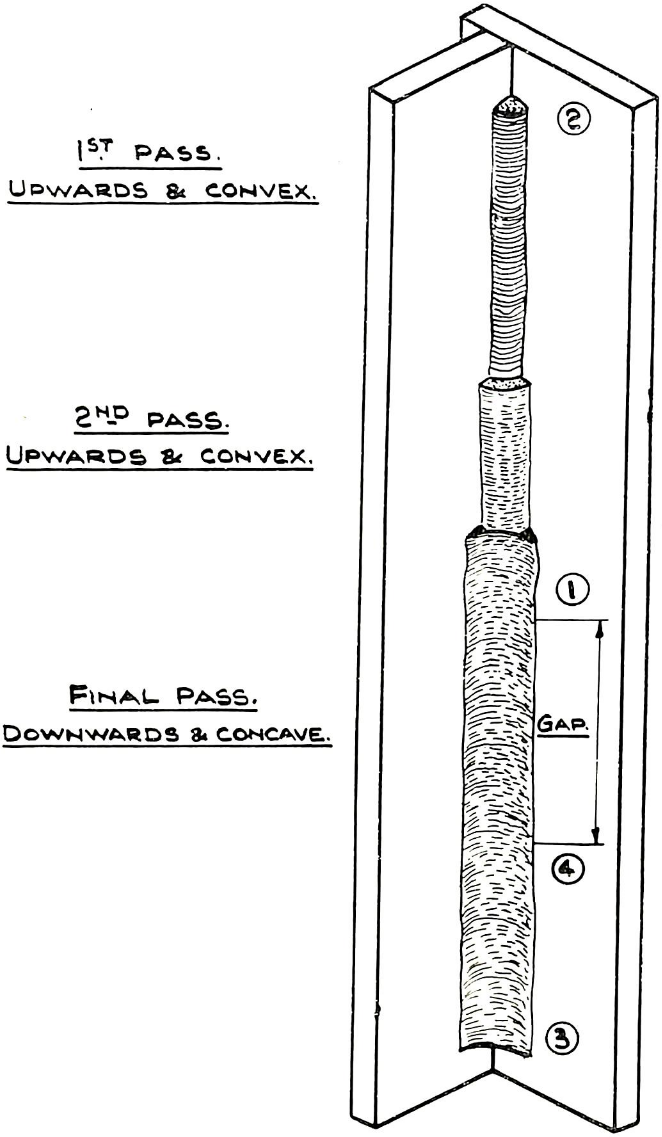
Overhead welds should always be laid in straight runs and no weaving should be attempted in load carrying passes. As the welds can only be deposited in narrow beads, multi-pass runs are invariably necessary. If appearance is of some importance, a final weave can be deposited over the weld, although this requires some skill on the part of the operator and in any case adds little strength to the weld.

Only welders of proved skill and efficiency should be employed in making overhead welded joints, and the passes should be laid in very short lengths, continually breaking the arc to allow the metal to freeze.

Typical faults in overhead welding are :—

(1) Undercutting due to the current being too high, the travel speed too low, or the electrode too large.

(2) Uneven leg length due to unequal play of the electrode on the two surfaces, often resulting in slag inclusions from later passes.



METHOD OF LAYING VERTICAL WELDS.

Fig. 12.

Inclined Welding.

The following general rules will usually be found to give a satisfactory procedure :—

(1) For seams inclined to the horizontal at an angle of less than 45° use downhand welds.
(2) For seams inclined to the horizontal at an angle greater than 45° use upward vertical welds.
(3) For overhead seams inclined to the horizontal at an angle of less than 45° use overhead welds.
(4) For overhead seams inclined to the horizontal at an angle greater than 45° use upward vertical welds.

All such cases must of course be judged on their merits, but the above division although somewhat rough and ready is a good general guide.

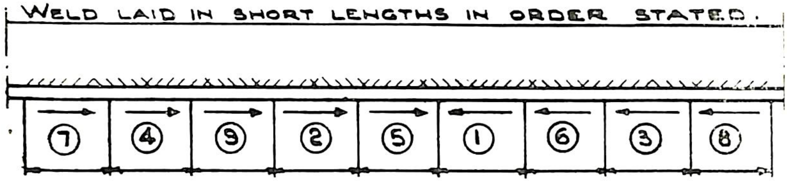
Penetration and Fusion.

Weld profiles indicated in the various diagrams are not, of course, intended to indicate the exact amount of penetration into the parent metal ; still less are they meant to infer a line of demarcation between parent metal and deposited metal. The whole principle of welding is to produce a complete fusion of parent metal and weld metal so that the parts joined together become in effect one single unit similar to a casting. For this purpose a deep penetration is not necessary and is in fact wasteful if the weld is required to have only normal weld strength. All that is required is to melt the outside layers of crystals in the parent metal to produce a proper fusion. When an unintentionally deep penetration has been obtained, it may mean that the current was unnecessarily too high.

Distortion.

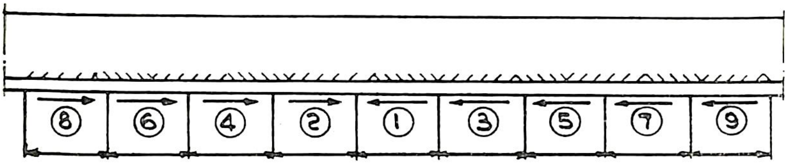
The distortion of component parts of a structure or member, produced mainly by continuous welding, constitutes one of the most difficult of all problems in welding practice. Distortion is due to the contraction of the molten metal in cooling, and unless adequate steps are taken to control the effect of such contraction it is likely to pull members out of shape to a prohibitive degree. With plates of large area, distortion is sometimes caused by internal stresses set up during the processes of manufacture, and this is naturally a point difficult to detect. Methods usually adopted to control distortion may be summarised as follows :—

(1) If distortion seems likely to occur, welding procedure is organised so that the pull of one weld cancels out the opposing pull of another weld.
(2) Individual runs are made comparatively short, compared with the thickness of the metal, as distortion effects are



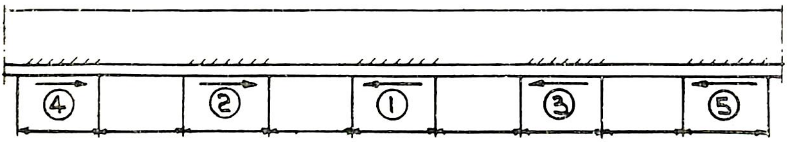
TYPICAL PROCEDURE TO AVOID DISTORTION.

Fig. 13.



ALTERNATIVE ARRANGEMENT OF WELDS (BACK STEPPING) TO AVOID DISTORTION.

Fig. 14.



ARRANGEMENT OF INTERMITTENT WELD TO AVOID DISTORTION.

Fig. 15.

cumulative and automatically increase with the lengthening of the weld.

(3) The parts should be rigidly tack welded at intervals of 3′ 0″ to 4′ 0″ before any runs of weld are laid, in order to keep individual parts in correct relative position.

(4) With continuous welds the weld metal may be deposited in short sections as shewn in Fig. 13. Alternatively what is known as the "back-stepping" method shewn in Fig. 14 may be used, or some variation of this. Where the welding is intermittent the procedure is as shewn in Fig. 15.

(5) The travel speed of welding should be as rapid as possible to prevent the heat moving ahead of the arc.

(6) The shape of the piece must be checked from time to time with a gauge during welding, and if distortion has commenced the work must be stopped on the weld causing the distortion and an opposing weld laid to bring the piece back to correct profile.

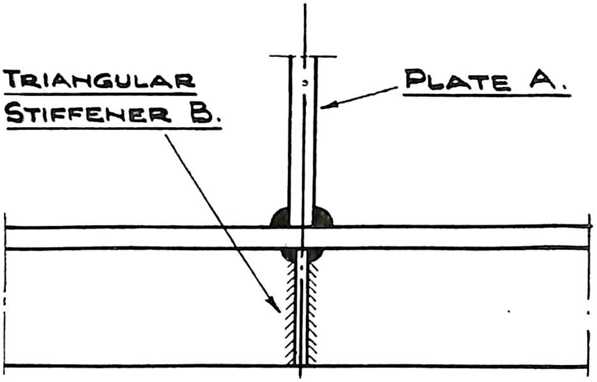
(7) Pre-heating of the plates is sometimes resorted to, so that the job expands and contracts as a whole, but this method is not sufficient in itself without additional safeguards.

(8) If it is expected that welding will produce distortion in any one direction, the plates may be given an initial distortion in the opposite direction before commencing welding, so that the welding may pull them back to the required alignment. This is a somewhat rough and ready procedure which may however meet the case where absolute accuracy of profile is not essential. The difficulty here of course is to assess how much distortion to allow for.

(9) When there is reason to believe that internal stresses have been set up in large plates during manufacture, distortion due to this cause may be prevented or minimised by re-rolling the plates before welding.

From the above remarks it will be clear that distortion provides a major problem which is not easily solved. It will also be apparent that welded work requires more expert supervision and experimental research than riveted work, and this point must be borne in mind when attempting to assess relative costs of manufacture.

Fig. 16 indicates a method of holding the legs of angles in correct alignment during welding. When plate A is welded on the angle may be distorted, and this can be prevented by first welding in the triangular stiffening plate B. Alternatively if the angle is large distortion can be avoided by welding on plate A by



METHOD OF PREVENTING
DISTORTION IN ANGLES.

Fig. 16.

the back-stepping method previously described, but this is not so satisfactory as the inclusion of the stiffening plate, which in addition to preventing distortion also gives a better distribution of the load, and maintains the casting analogy referred to previously.

One of the most troublesome distortion problems is provided by the angular distortion of flange plates in welded plate girders. It will be clear that the deflection at the edges of the plate will be approximately proportional to the width of the plate and to the fillet size, and inversely proportional to the thickness of the flange plate. Working on this basis it has been found in practice that if a $\frac{5}{16}''$ fillet weld is applied to a $12'' \times 1''$ flange plate the distortion is practically nil. From this a simple rule may be devised.

Let w = width of flange plate in inches.
t = thickness of flange plate in inches.
s = fillet size in inches.

Then for the above $12'' \times 1''$ plate with $\frac{5}{16}''$ fillets

$$\frac{w \times s}{t} = \frac{12 \times \frac{5}{16}}{1} = 3{\cdot}75$$

Thus in any plate girder where this ratio equals or nearly equals 3·75, the distortion will be small. This means in effect that a small adjustment in plate width and thickness will often solve this particular problem without having to resort to special devices.

(Further useful information on the control of distortion may be obtained from the article on "Aspects of Welding Distortion in Shipbuilding," by D. M. Kerr, in *Transactions of the Institute of Welding*, dated October, 1950, and from A.E.S.D. pamphlet "The Control of Distortion in Welded Structures," by E. W. Ansell).

PART II.

DESIGN OF DETAILS.

Connection of Angles.

Case 1—Full Area Effective.

Referring to Fig. 17 :—

Let A = area of section of angle in sq. ins.
F = permissible stress in angle in tons/sq. in.
P = load carried by angle = AF.
S_1 = load in weld at heel of angle in tons.
S_2 = load in weld at toe of angle in tons.
f = strength of weld in tons per lineal inch.

Taking moments about X (the toe of the angle)

$$S_1 = \frac{Pb}{a+b}$$

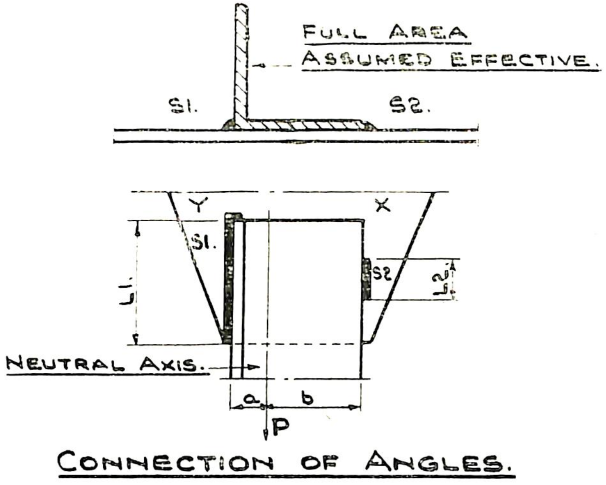


Fig. 17.

Taking moments about Y (the heel of the angle)

$$S_2 = \frac{Pa}{a+b}$$

Using fillets of equal size on both sides.

Effective length of weld S_1

$$= L_1 = \frac{S_1}{f} = \frac{Pb}{f\,(a+b)} = \frac{P}{f}\left(\frac{b}{a+b}\right)$$

Effective length of weld S_2

$$= L_2 = \frac{S_2}{f} = \frac{Pa}{f\,(a+b)} = \frac{P}{f}\left(\frac{a}{a+b}\right)$$

If for constructional reasons it is desired to have equal lengths of weld at heel and toe :—

Strength per lineal inch required at heel

$$= \frac{S_1}{L} = \frac{Pb}{L\,(a+b)} = \frac{P}{L}\left(\frac{b}{a+b}\right)$$

Strength per lineal inch required at toe

$$= \frac{S_2}{L} = \frac{Pa}{L\,(a+b)} = \frac{P}{L}\left(\frac{a}{a+b}\right)$$

Thus it will be seen that whether it is desired to find the lengths of equal sized welds or the sizes of weld required for equal lengths, in either case the required values have the same ratio to each other as dimension b has to dimension a.

Case 2—Assuming only half area of outstanding leg is effective in taking load.

Referring to Fig. 18.

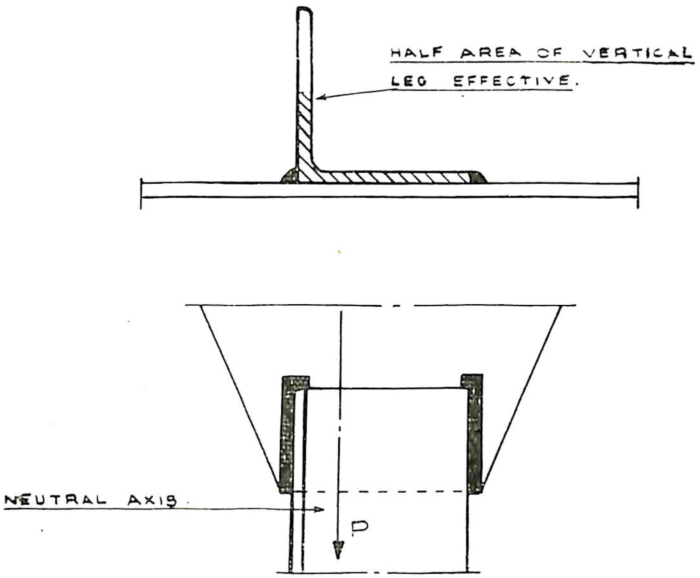
Let A = assumed effective area of angle in sq. ins.
= area of horizontal leg + half area of vertical leg.
F = permissible stress in angle in tons/sq. in.
P = load carried by angle = AF.
f = strength of weld in tons per lineal inch.

Using fillets of the same size on both sides :—

Effective length of weld required each side $= \dfrac{P}{2f}$

This procedure is to some extent approximate as it assumes that the effective area of the angle is divided into two equal parts by the neutral axis. The degree of error is very small, especially in the case of small angles.

If it is desired to be exact the position of the neutral axis for the assumed effective area of the angle must be found, and the procedure outlined for Case 1 adopted.



CONNECTION OF ANGLES.

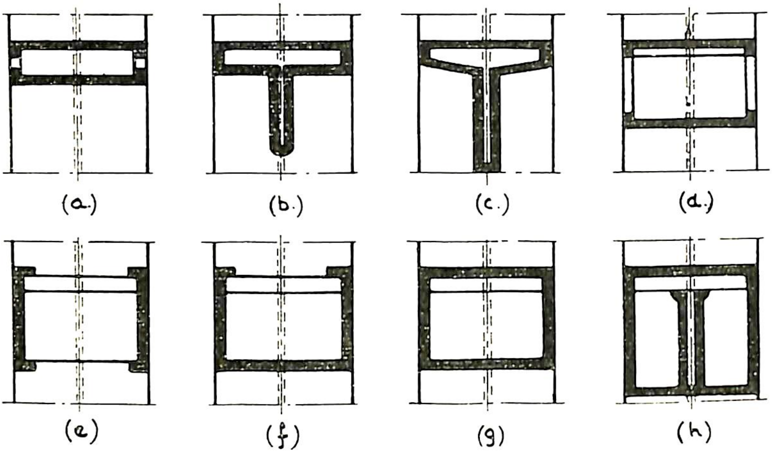
Fig. 18.

If side fillets alone are incapable of transmitting the load in the angle, then either one or two end fillets must also be included. Typical examples will be found later in the case of a welded roof truss.

Beam Seatings on Stanchions.

With riveted construction seatings consist of either a cleat or a built up bracket according to the load to be transmitted. If the stanchion were a casting the seating would consist of a lug cast on with or without a stiffener beneath to suit the load. Maintaining the principle that welded work is analogous to a casting rather than to riveted work, a lug consisting of a flat about $\frac{1}{2}''$ thick, as shewn in Fig. 19 (a), can be used if the seating is purely at rest, taking no load.

Such a seating can also be used in place of a riveted seating cleat when a beam reaction is transferred through web cleats with rivets or bolts. If the seating transmits load any of the constructions shewn in Figs. 19 (b) to (h) can be used according to requirements. Fig. 19 (b) shews a tee-seating which is quite suitable for small loads. Fig. 19 (c) shews a joist cutting for larger loads, and for economy the depth of bracket should be half



BEAM SEATINGS.

Fig. 19.

the depth of joist used so that two brackets can be obtained from the same piece of joist with a single cut. Figs. 19 (d), (e), (f) and (g) shew cleat seatings with either side or end fillets, or both, according to the load to be carried. Fig. 19 (h) shews a deeper cleat with a stiffener for heavier loads. In all these cases the effective length of weld connecting the seating to the stanchion should be based on an allowable stress of $6\frac{1}{2}$ tons per square inch when designing to B.S. 449, 1948, the area of section of the fillet being the effective length multiplied by the throat thickness.

Example :—Using $\frac{1}{4}''$ fillets, find the effective length of weld required to transmit a reaction of 20 tons.

$$\text{Effective weld area required} = \frac{20}{6{\cdot}5} = 3{\cdot}077 \text{ sq. ins.}$$

$$\text{Effective length required} = \frac{3{\cdot}077}{\frac{1}{4} \times 0{\cdot}707} = 17{\cdot}4'' \text{ (say } 18'').$$

Eccentric Beam Seatings.

Fig. 20 gives a typical example of such a seating. The load is first transferred from the cleat to the gusset and thence to the stanchion flange. If the top edge of the gusset is machined to give a perfect bearing for the beam some of the load will be transferred directly into the gusset, but it seems safer to assume that the welds connecting the cleat to the gusset must take the full load. On this basis there are three stages in design :—

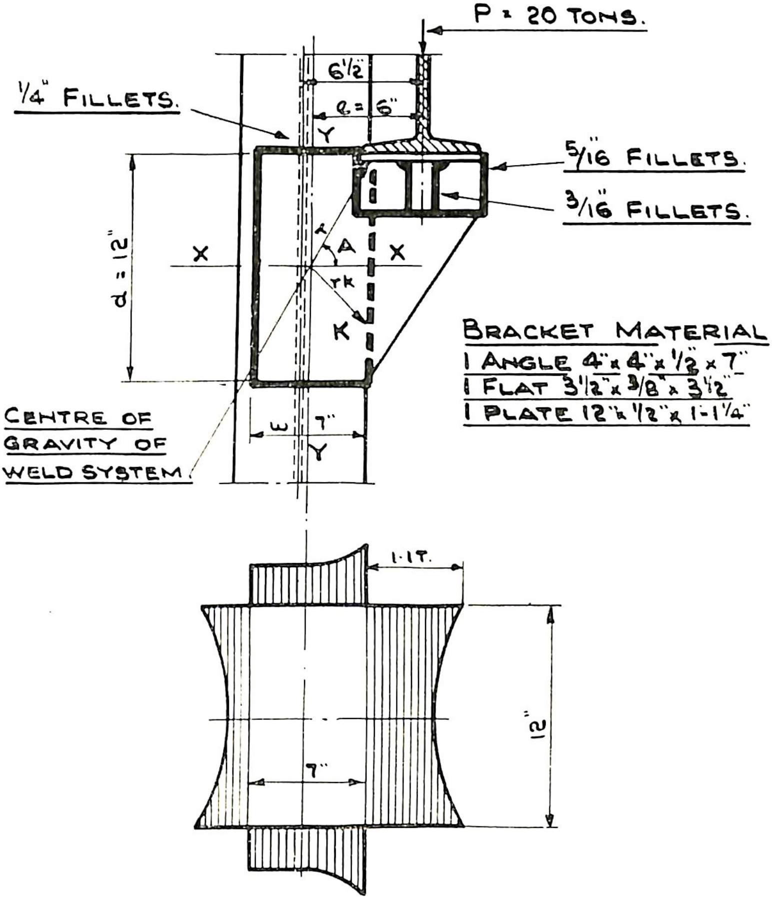


DIAGRAM OF STRESS DISTRIBUTION IN ¼" FILLETS.

ECCENTRIC BEAM SEATING.

Fig. 20.

(1) The welds connecting the cleat to the gusset take the load in shear.

(2) The gusset must have sufficient depth and thickness to transmit the load.

(3) The welds connecting the gusset to the stanchion must be of sufficient strength to take the direct shear and the moment produced by the load.

Connection of Cleat to Gusset.

Try $4'' \times 4'' \times \frac{1}{2}''$ angle $7''$ long with $\frac{5}{16}''$ fillets.
Effective length of weld $= 4'' + 4'' + 7'' - (2 \times \frac{5}{16}'') = 14\frac{3}{8}''$.
Allowable load per inch for $\frac{5}{16}''$ fillet at $6\frac{1}{2}$ tons per. sq in.
$= 1{\cdot}44$ tons.
Total value of weld $= 14\frac{3}{8} \times 1{\cdot}44 = 20{\cdot}7$ tons.
This is satisfactory.

The triangular stiffener inside the cleat is intended to prevent bending in the horizontal leg.

Gusset Strength.

The gusset must have sufficient vertical sectional area immediately beneath the beam centre line to transmit half of the load in shear, and sufficient vertical sectional area at the edge of the cleat nearest to the stanchion to transmit the whole of the load in shear. Usually unless the load is very large and/or the gusset has too sharp a cutaway at the top outside corner this requirement is covered by the construction, but it should be checked.

In addition the gusset must have a sufficient vertical section modulus on line YY to resist the moment. Checking first for bending.

Moment of load about YY $= 20 \times 6 = 120$ tons ins.

$$Z \text{ of gusset} = \frac{BD^2}{6} = \frac{\frac{1}{2} \times 12^2}{6} = 12 \text{ ins.}^3$$

$$fb = \frac{120}{12} = 10 \text{ tons per sq. in.}$$

This is satisfactory, especially as the gusset at this section and for some distance in advance is welded to the stanchion flange.

Checking for Shear.

Area of gusset $= 12 \times \frac{1}{2} = 6$ sq. ins.

$$fs = \frac{20}{6} = 3{\cdot}33 \text{ tons per sq. in.}$$

This also is satisfactory and the gusset will be suitable.

Welds Connecting Gusset to Stanchion.

Calculations will be made on the basis of 1″ throat thickness. It will be found in practice that this method is a convenient procedure.

Moment of load about centre of gravity of weld system

$$= 20 \times 6 = 120 \text{ tons ins.}$$

Depth d of vertical welds $= 12''$.
Width w of horizontal welds $= 7''$.

Distance r from centre of gravity to extreme fibre of welds

$$= \sqrt{3\tfrac{1}{2}^2 + 6^2} = 6{\cdot}95''.$$

Total length of weld $= 2\,(7+12) = 38''$.

Ip = polar moment of inertia $= I_{xx} + I_{yy}$.

$$\text{For 1'' throat thickness } Ip = \left(\frac{2 \times 12^3}{12}\right) + \left(\frac{2 \times 7^3}{12}\right)$$

$$+ (2 \times 12 \times 3\tfrac{1}{2}^2) + (2 \times 7 \times 6^2) = 1143 \text{ ins.}^4.$$

$$\text{Shear stress per inch run of weld} = \frac{20}{38} = 0{\cdot}53 \text{ ton.}$$

$$\text{Bending stress per inch run of weld} = \frac{M \times r}{Ip} = \frac{120 \times 6{\cdot}95}{1143}$$

$$= 0{\cdot}73 \text{ tons.}$$

Resultant stress per inch run of weld

$$= \sqrt{0{\cdot}53^2 + 0{\cdot}73^2 + (2 \times 0{\cdot}53 \times 0{\cdot}73 \times \cos A)} = 1{\cdot}1 \text{ tons.}$$

$\frac{1}{4}''$ fillets at 6·5 tons per square inch are capable of taking a load of 1·15 tons per inch run, and will be used.

It will be noted from the diagram of stress distribution that the load of 1·1 tons per inch is a maximum, occurring only at the extreme corners, and elsewhere it will be less. For example, at any point K, at a distance of rk from the centre of gravity, the bending stress will be $\dfrac{M \times rk}{Ip} = \dfrac{120 \times rk}{1143}$

This value is less than the maximum bending stress previously calculated, so that the load per inch run at K will be less than 1·1 tons.

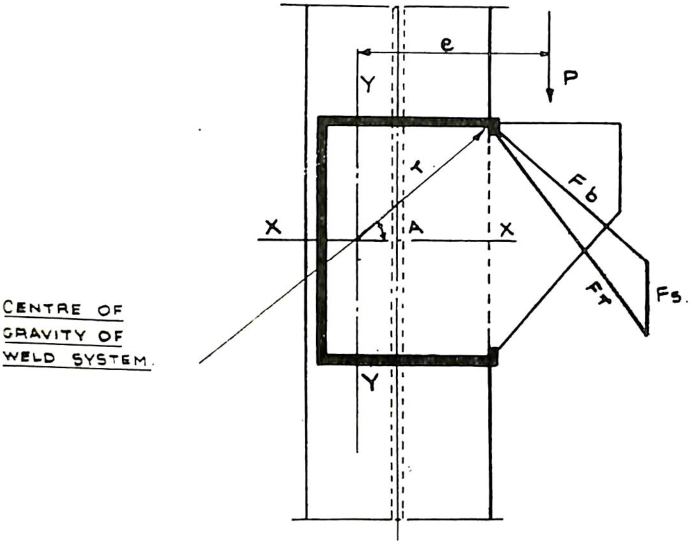
The procedure given above for the calculation of the load in the welds connecting the gusset to the stanchion is the method generally adopted in structural practice, but is open to some criticism. It is argued that the welds are subject to torsional stresses rather than bending stresses, produced by a torque moment of $20 \times 6 = 120$ tons ins. On this basis, with welds of the same size throughout the shear stress due to torque will be constant throughout, or very nearly so, amounting to (for 1″ throat)

$$\frac{120}{2 \times 12 \times 7 \times 1} = 0{\cdot}714 \text{ tons per sq. in.}$$

Maximum shear stress in the weld would thus be 0·714 + 0·53 (due to vertical shear) = 1·244 tons per sq. in.

This value is more than the previous result and if this procedure is adopted a $\frac{5}{16}$″ fillet would be required.

Generally it will be found that with small loads and/or small eccentricities the same size of weld will be required whichever method is adopted, but as loads or eccentricities increase there will be a variation in result. It therefore seems desirable, until such time as welding practice is more generally established, to design for safety as an increase of $\frac{1}{16}$″ in the fillet size will not greatly increase the cost.



ECCENTRIC BEAM SEATINGS.

SPECIAL CASES WITH WELDS ON 3 SIDES ONLY.

Fig. 21.

Eccentric Beam Seating (Special Case).

Fig. 21 is generally similar in construction to Fig. 20, the only variation being that the gusset is welded to the stanchion flange on three sides only. The centre of gravity of the weld system is first found and the procedure is then as before. It should be noted that the eccentricity is greater than in Fig. 20, giving an increased moment, so that with less weld metal in addition a deeper bracket will be required for the same load. Unless therefore there are good constructional reasons for omitting the additional run of weld there seems no point in preferring this type of bracket to that in Fig. 20.

Crane Brackets to Stanchions.

Figs. 22 to 25 shew typical constructions for the more usual types of brackets. For simplicity, calculations are based on a throat thickness of 1″, as in previous examples.

Referring to Fig. 22 and assuming effective length of weld is 6″.

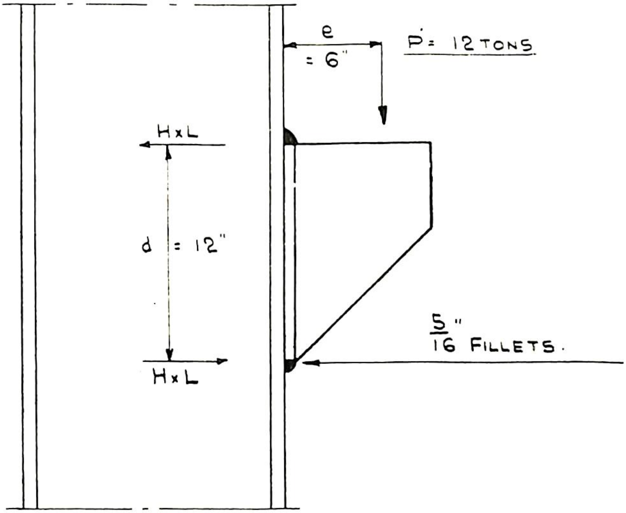
$$I_{xx} \text{ of welds} = 2Ar^2 = 2 \times 6 \times 1 \times 6^2 = 432 \text{ ins.}^4$$

$$Z_{xx} \text{ of welds} = \frac{I}{y} = \frac{432}{6} = 72 \text{ ins.}^3$$

$$fb = \frac{M}{Z} = \frac{P \times e}{Z} = \frac{12 \times 6}{72} = 1 \text{ ton per sq. in.}$$

$$fs = \frac{P}{A} = \frac{12}{2 \times 6 \times 1} = 1 \text{ ton per sq. in.}$$

$$\text{Resultant stress} = \sqrt{1^2 + 1^2} = 1{\cdot}414 \text{ tons per sq. in.}$$



CRANE BRACKETS WITH END FILLETS.

Fig. 22.

An alternative procedure, which is actually the same procedure written down in different form, is as follows :—

Let H = horizontal component of load in weld per inch run.
l = effective length of weld = 6″.
V = vertical component of load in weld per inch run.

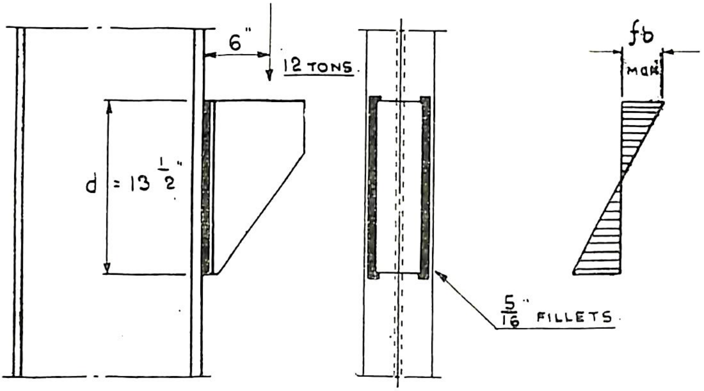
$$M = P \times e = H \times l \times d.$$

$$\therefore \quad H = \frac{P \times e}{l \times d} = \frac{12 \times 6}{6 \times 12} = 1 \text{ ton per inch.}$$

$$V = \frac{P}{2l} = \frac{12}{2 \times 6} = 1 \text{ ton per inch.}$$

$$\text{Resultant} = \sqrt{1^2 + 1^2} = 1{\cdot}414 \text{ tons per inch.}$$

$\frac{5}{16}$″ fillets at $6\frac{1}{2}$ tons per square inch will take a load of 1·44 tons per inch run and will be used.



CRANE BRACKETS WITH SIDE FILLETS.

Fig. 23.

For Fig. 23 the procedure is as follows :—

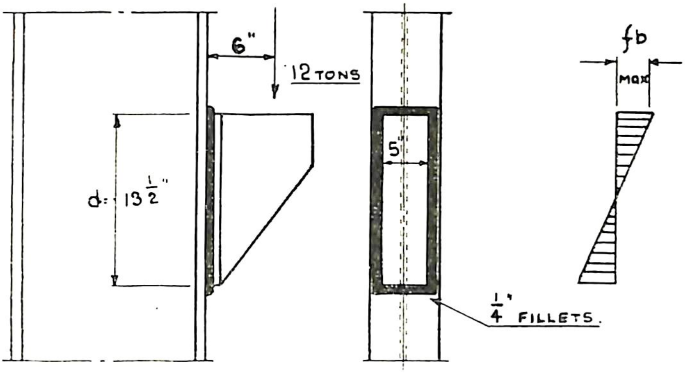
$$Z_{xx} \text{ of welds} = \frac{2\,BD^2}{6} = \frac{2 \times 1 \times 13{\cdot}5^2}{6}$$

$$fb = \frac{M}{Z} = \frac{6 \times 12 \times 6}{2 \times 1 \times 13{\cdot}5^2} = 1{\cdot}19 \text{ tons per sq. in.}$$

$$fs = \frac{P}{A} = \frac{12}{2 \times 13{\cdot}5 \times 1} = 0{\cdot}44 \text{ tons per sq. in.}$$

$$\text{Resultant} = \sqrt{1{\cdot}19^2 + 0{\cdot}44^2} = 1{\cdot}27 \text{ tons per sq. in.}$$

$\frac{5}{16}$″ fillets taking a load of 1·44 tons per inch run will be used.



CRANE BRACKETS WITH SIDE AND END FILLETS.

Fig. 24.

For Fig. 24 the procedure is as follows :—

I_{xx} of top and bottom welds $= 2 \times 5 \times 1 \times 6{\cdot}75^2 \qquad = 455{\cdot}6 \text{ ins.}^4$

I_{xx} of side welds $= \dfrac{2 \times 1 \times 13{\cdot}5^3}{12} \qquad = 410{\cdot}06 \text{ ins.}^4$

Total $I_{xx} = 455{\cdot}6 + 410{\cdot}06 \qquad = 865{\cdot}66 \text{ ins.}^4$

Z_{xx} of welds $= \dfrac{I}{y} = \dfrac{865{\cdot}66}{6{\cdot}75} \qquad = 128{\cdot}3 \text{ ins.}^3$

$fb = \dfrac{M}{Z} \quad \dfrac{12 \times 6}{128{\cdot}3} = 0{\cdot}56$ tons per sq. in.

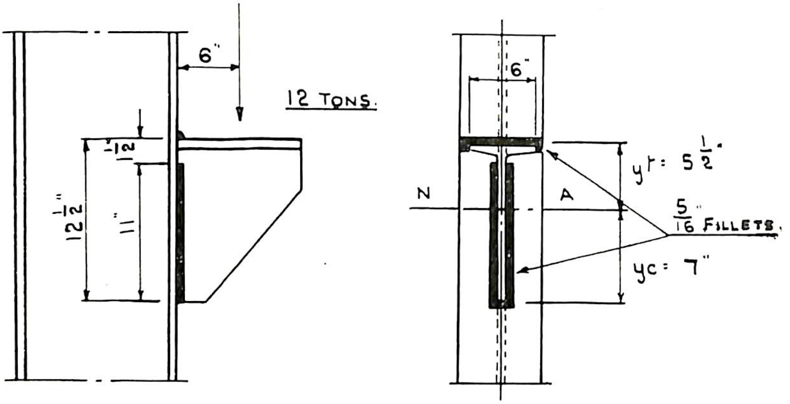
Total length of weld $= 2\,(5 + 13{\cdot}5) = 37$ ins.

$fs = \dfrac{P}{A} = \dfrac{12}{37 \times 1} = 0{\cdot}324$ tons per sq. in.

Resultant $= \sqrt{0{\cdot}56^2 + 0{\cdot}324^2} = 0{\cdot}65$ tons per sq, in.

$\frac{3}{16}$" fillets at $6\frac{1}{2}$ tons per square inch will take a load of 0·86 ton per lineal inch but these are rather small for a joist flange and $\frac{1}{4}$" fillets will be used.

For purposes of comparison this example has been worked out for a bracket of the same depth as that in Fig. 23, and it will be seen that the additional fillets at the top and bottom give considerable additional strength. With these fillets the bracket could in fact be reduced in depth.



UNSYMMETRICAL CRANE BRACKETS.

Fig. 25.

For Fig. 25 the procedure is as follows :—

To find the centre of gravity of the weld system, moments are taken about the top flange of the bracket.

Moment of flange weld $= 6 \times 1 \times 0 = 0$

Moment of web welds $= 2 \times 11 \times 1 \times 7 = 154$ ins.^{3}

Area of welds $= (6 + 11 + 11) \times 1 = 28$ sq. ins.

$$y_t = \frac{154}{28} = 5\tfrac{1}{2}''. \qquad y_c = 12\tfrac{1}{2}'' - 5\tfrac{1}{2}'' = 7 \text{ ins.}$$

$$I_{xx} = (6 \times 1 \times 5\tfrac{1}{2}^2) + \left(\frac{2 \times 1 \times 11^3}{12}\right) + (2 \times 11 \times 1 \times 1\tfrac{1}{2}^2)$$

$$= 452{\cdot}8 \text{ ins.}^4$$

$$Z_{xxt} = \frac{452{\cdot}8}{5\frac{1}{2}} = 82{\cdot}3 \text{ ins.}^3 \quad Z_{xxc} = \frac{452{\cdot}8}{7} = 64{\cdot}7 \text{ ins.}^3$$

$$fb_t = \frac{M}{Z_t} = \frac{72}{82{\cdot}3} = 0{\cdot}87 \text{ tons per sq. in.}$$

$$fb_c = \frac{M}{Z_c} = \frac{72}{64{\cdot}7} = 1{\cdot}11 \text{ tons per sq. in.}$$

$$fs = \frac{P}{A} = \frac{12}{28} = 0{\cdot}43 \text{ tons per sq. in.}$$

Resultant stress at extreme tension fibre (top of bracket)

$$= \sqrt{0{\cdot}87^2 + 0{\cdot}43^2} = 0{\cdot}97 \text{ tons per sq. in.}$$

Resultant stress at extreme compression fibre (bottom of bracket)

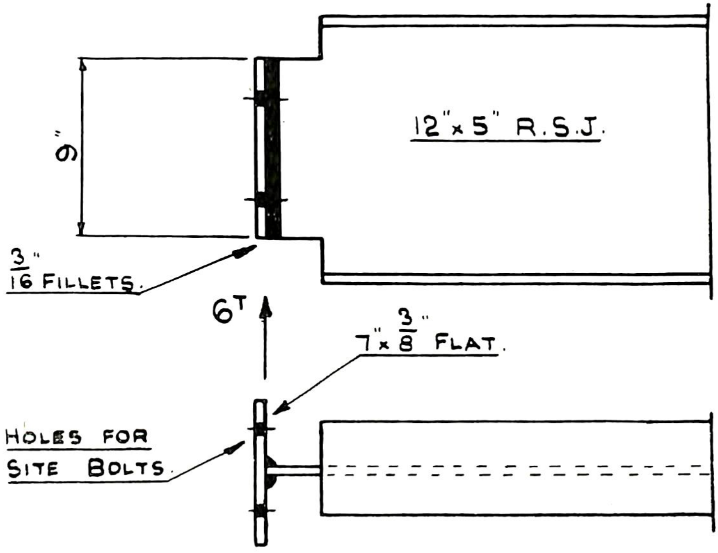
$$= \sqrt{1{\cdot}11^2 + 0{\cdot}43^2} \quad = \quad 1{\cdot}19 \text{ tons per sq. in.}$$

At $6\frac{1}{2}$ tons per square inch $\frac{1}{4}''$ fillets will take a load of 1·15 tons per lineal inch and this would do for the flange weld, but as it is rather fine for the web welds, all welds will be made $\frac{5}{16}''$ fillet for uniformity.

Beam End Connections (Non-Rigid).

Fig. 26 illustrates a typical beam end detail which is suitable for practically any beam section, and compares very favourably with an all-riveted or bolted connection involving the use of web cleats. The detail is, in fact, a typical example of how shop welding can be usefully and economically introduced into building work where site connections are bolted or riveted. There is very little eccentricity of load on the welds, which may therefore be designed for simple shear.

Depth of web at end $= 9''$.
Assume effective depth of fillets $= 8\frac{1}{2}''$.
Reaction $= 6$ tons.



BEAM CONNECTIONS (NON-RIGID)
SHOP WELDED.
Fig. 26.

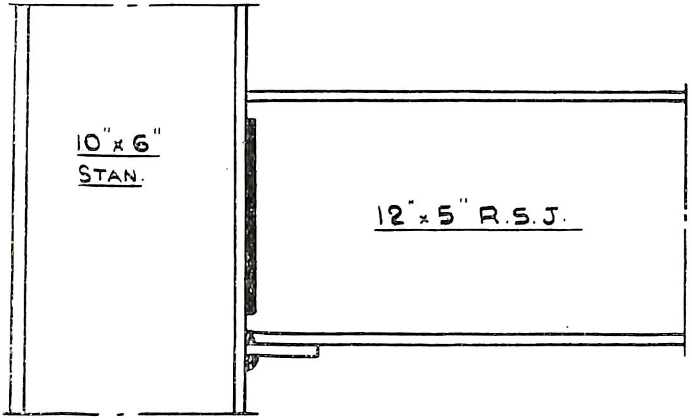
$$\text{Load per inch run of fillet} = \frac{6}{2 \times 8\frac{1}{2}} = 0{\cdot}352 \text{ ton.}$$

At $6\frac{1}{2}$ tons per square inch $\frac{1}{8}''$ fillets have a value of 0·57 tons per lineal inch, but these are rather small and $\frac{3}{16}''$ fillets will be used, which will also give ample security against moment due to the slight eccentricity. End plates will be made the same thickness as the beam web, with a sufficient width to insert the site bolts.

For a connection of this type it is not essential to make the fillets continuous. Where the beam is deep and the reaction small, welds can be intermittent with a sufficient total effective length to take the reaction.

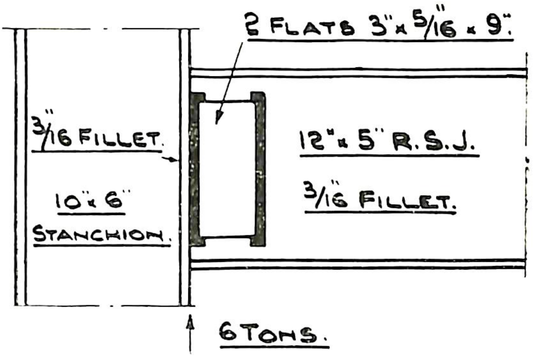
Fig. 27 shews an alternative type of beam connection involving site welding. Site welding above ground level is nearly always a hazardous operation in addition to being expensive, and there appears to be no adequate reason for introducing this type of connection into a structure in preference to that shewn in Fig. 26. The lug on the stanchion serves purely as a rest, and the fillets to the beam web are designed to take the reaction in simple shear as in the case of Fig. 26.

A further alternative end detail is shewn in Fig. 28, which also involves site welding. The flats are shop-welded to the beam and site-welded to the stanchion. It is assumed that the welds share the moment produced by the eccentricity of the connection.



BEAM CONNECTIONS (NON-RIGID) SITE WELDED

Fig. 27.



BEAM CONNECTIONS (NON-RIGID) SITE WELDED.

Fig. 28.

Moment from reaction $= 6 \times 3 = 18$ tons ins.

Moment taken by each pair of fillets $= \dfrac{18}{2} = 9$ tons. ins.

For 1" throat thickness Z of each pair of fillets

$$= \frac{2 \times 1 \times 9^2}{6} = 27 \text{ ins.}^3$$

$$fb = \frac{M}{Z} = \frac{9}{27} = 0{\cdot}33 \text{ tons per sq. in.}$$

$$fs = \frac{6}{2 \times 9 \times 1} = 0{\cdot}33 \text{ tons per sq. in.}$$

Resultant $= 0{\cdot}33\sqrt{2} = 0{\cdot}47$ tons per sq. in.

$\frac{3}{16}$" fillets at $6\frac{1}{2}$ tons per square inch will take 0·86 tons per lineal inch and will be used.

Rigid Beam End Connections.

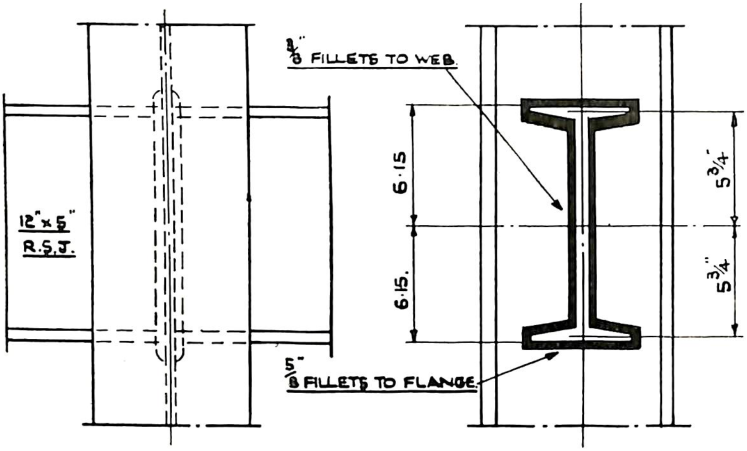
Type 1—Beams to Stanchion Webs (Fig. 29).

The general principle of design for connections of this type is to make the welds as strong as the beam. So far as loads are concerned this usually means erring slightly on the strong side, but it is a sound principle that connections generally should have the same margin of strength as the members.

Moment of inertia of beam $= 221$ ins.^{4}

Moment of resistance of beam at 10 tons per sq. in.

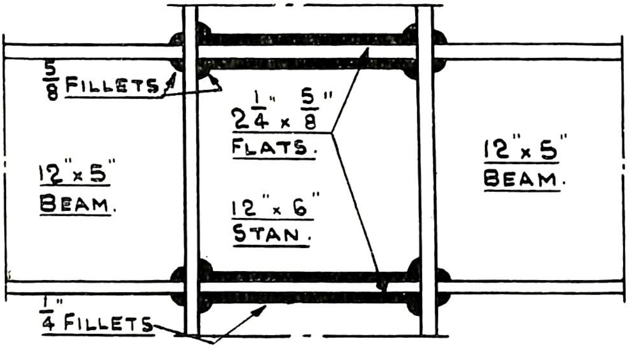
$$= \frac{fI}{y} = \frac{10 \times 221}{6} = 368{\cdot}3 \text{ tons ins.}$$



BEAM CONNECTION TO STANCHION WEB.

(RIGID TYPE) SITE WELDED.

Fig. 29.



BEAM CONNECTIONS TO STANCHION FLANGES (RIGID TYPE) SITE WELDED.

Fig. 30.

To obtain a moment of resistance of 368·3 tons ins. in the welds for a stress of $6\frac{1}{2}$ tons per square inch, moment of inertia required

$$\text{in welds} = \frac{368{\cdot}3 \times y}{f} = \frac{368{\cdot}3 \times 6{\cdot}15}{6{\cdot}5} = 348{\cdot}5 \text{ ins.}^4$$

Assume $\frac{5}{8}''$ fillets to flange with throat thickness of 0·442″.

Moment of inertia of flange fillets $= 2ar^2 = 2 \times 10\frac{1}{2} \times 0{\cdot}442 \times 5\frac{3}{4}^2$
$= 306{\cdot}9 \text{ ins.}^4$

Moment of inertia required in web fillets $= 348{\cdot}5 - 306{\cdot}9$
$41{\cdot}6 \text{ ins.}^4$

For $\frac{3}{8}''$ fillets with throat thickness of 0·265″, moment of inertia

$$\text{given} = \frac{2BD^3}{12} = \frac{2 \times 0{\cdot}265 \times 10^3}{12} = 44{\cdot}2 \text{ ins.}^4$$

Use $\frac{5}{8}''$ fillets to outside and inside of both flanges, and $\frac{3}{8}''$ fillets to web.

Type 2—Beams to Stanchion Flanges—Connections on Both Flanges (Fig. 30).

The procedure is to assume that the maximum permissible bending moment is developed in the connections assuming the maximum beam stress of 10 tons per square inch and the maximum weld stress of $6\frac{1}{2}$ tons per square inch. On this basis the connection is as strong as the beam.

Thickness of beam flange $= 0{\cdot}55''$.
Maximum load in flange $= 5 \times 0{\cdot}55 \times 10 = \pm 27{\cdot}5$ tons.

These forces must be carried across the stanchion by means of the welds and stiffening flats.

$$\text{Load per lineal inch of flange weld} = \frac{27{\cdot}5}{10{\cdot}5} = 2{\cdot}62 \text{ tons.}$$

$\frac{5}{8}''$ fillets at $6\frac{1}{2}$ tons per square inch will take 2·87 tons per lineal inch and will be used.

The purpose of the stiffening flats is to stiffen the flanges against deformation and also to ensure uniformity in the weld stresses. Assuming all the load is transmitted through the stiffening flats (actually they will receive slightly less), area required in flats

$$= \frac{27{\cdot}5}{10} = 2{\cdot}75 \text{ sq. ins.}$$

2 flats $2\frac{1}{4}'' \times \frac{5}{8}''$ will be used. Area given $= 2{\cdot}81$ sq. ins.

The welds at the ends of the stiffening flats must also transmit the load, and $\frac{5}{8}''$ fillets will be used as before.

The stiffeners need to be tack welded to the stanchion web and $\frac{1}{4}''$ fillets (decided without calculation) will serve for this purpose.

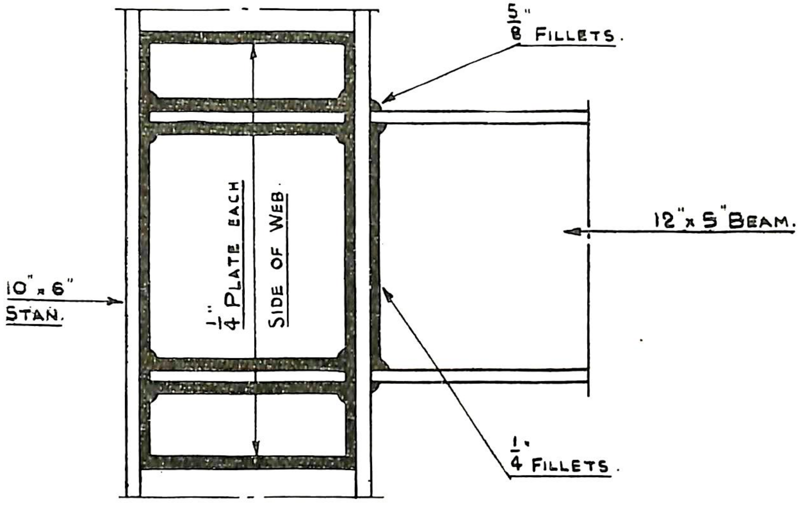
Type 3—Beam to Stanchion Flange—One Side Only (Fig. 31).

This case is generally similar to Type 2, requiring similar treatment except that the stanchion web now needs stiffening to resist shear and possibly buckling. Welds and stiffening flats to the stanchion flanges will be as previously, and additional calculations are necessary only for checking the strength of the stanchion web.

Load from beam flange (as previously) = 27·5 tons.

Assuming all this load is transmitted into the stanchion web,

$$\text{shear stress in web} = \frac{27{\cdot}5}{10 \times 0{\cdot}4} = 6{\cdot}88 \text{ tons per sq. in.}$$



BEAM CONNECTION TO STANDARD FLANGE (RIGID TYPE) SITE WELDED.

Fig. 31.

This is higher than the allowable shear stress of 6·5 tons, and although the excess is not very great it seems desirable to stiffen the web against shear.

To illustrate the procedure, the web will also be checked for buckling.*

The buckling length is measured along the vertical centre line of the stanchion and the load from the beam flange is assumed to spread at an angle of 45° both upwards and downwards to the stanchion centre line. The buckling length will thus equal the thickness of the beam flange, plus the size of the two $\frac{5}{8}''$ welds above and below, plus the depth of the stanchion section, *i.e.*,

---

*For detailed explanation of method of checking beam webs against buckling and also bearing, see A.E.S.D. pamphlet "Structural Steelwork Design to B.S.S. 449 (1948)."

$$0{\cdot}55 + \tfrac{5}{8} + \tfrac{5}{8} + 10 = 11{\cdot}8''.$$

Ratio of slenderness for buckling stress

$$= \frac{d\sqrt{3}}{t} = \frac{8{\cdot}14 \times 1{\cdot}73}{0{\cdot}4} = 35$$

Allowable buckling stress (Table 7 of B.S. 449) = 7·3 tons/sq. in.

Allowable buckling load = $7{\cdot}3 \times 11{\cdot}8 \times 0{\cdot}4$ = 34·5 tons.

The web will also be checked against bearing. The bearing line is at the bottom of the stanchion flange root, that is 1·43 inches from the flange face, and the load is assumed to spread at an angle of 30° both upwards and downwards to this line.

$$\text{Bearing length} = 0{\cdot}55'' + \tfrac{5}{8}'' + \tfrac{5}{8}'' + \frac{2 \times 1{\cdot}43}{\tan 30^\circ} = 6{\cdot}8''.$$

$$\text{Bearing stress} = \frac{27{\cdot}5}{6{\cdot}8 \times 0{\cdot}4} = 10{\cdot}11 \text{ tons per sq. in.}$$

This is less than the allowable bearing stress of 12 tons per square inch and the web is safe for bearing.

The necessary stiffening against shear may be obtained by inserting a $\frac{1}{4}''$ flat on each side of the web securely welded to the profile of the flange and to the web.

$$\text{Shear stress in stiffened web} = \frac{27{\cdot}5}{10 \times 0{\cdot}9} = 3{\cdot}06 \text{ tons/sq., in.}$$

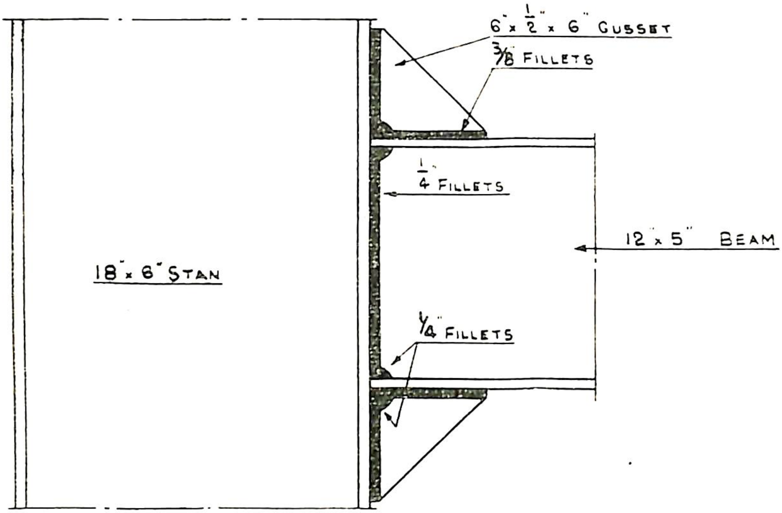
When the depth of the stanchion is considerably more than that of the beam, that is to say, sufficiently more to make web stiffening unnecessary, a slightly different procedure is often adopted which also dispenses with the stanchion flange stiffeners. The procedure is shewn in Fig. 32, and the beam together with the brackets may be considered as a casting loaded somewhat similarly to a stanchion base subject to bending moment. It will be clear that a small part of the load from the beam flanges, assumed previously to be transmitted directly to the stanchion, will now be transferred through the beam web, and it will be assumed in this case that one-third of the flange load will be thus transferred (a safe assumption) leaving two-thirds to be transferred through the brackets.

Load transferred through web = $\frac{1}{3} \times 27{\cdot}5$ = 9·2 tons.

Load transferred through bracket = $\frac{2}{3} \times 27{\cdot}5$ = 18·3 tons.

Sectional area of gusset = $6 \times \frac{1}{2} \times 0{\cdot}707$ = 2·12 sq. ins.

$$\text{Stress in gusset} = \frac{18{\cdot}3}{2{\cdot}12} = 8{\cdot}63 \text{ tons per sq. in.}$$



BEAM CONNECTION TO STANCHION FLANGE (RIGID TYPE) SITE WELDED

Fig. 32.

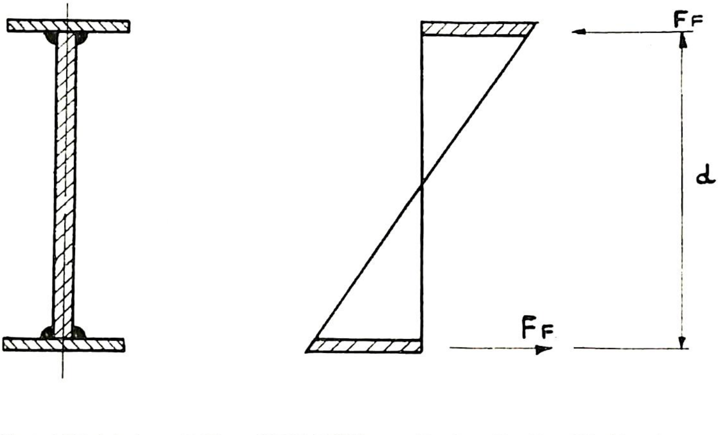


DIAGRAM OF STRESS DISTRIBUTION.

Fig. 33.

$$\text{Load per inch run of weld} = \frac{18{\cdot}3}{2 \times 6} = 1{\cdot}53 \text{ tons.}$$

$\frac{3}{8}''$ fillets at $6\frac{1}{2}$ tons per square inch will take 1·72 tons per lineal inch and will be used.

The weld between the beam flange and the stanchion should be designed (for safety) to take 9·2 tons, although part of this will undoubtedly go through the beam web fillets.

$$\text{Load per inch run of weld} = \frac{9{\cdot}2}{\text{(say) } 9} = 1{\cdot}02 \text{ tons.}$$

$\frac{1}{4}''$ fillets will take 1·15 tons per inch and will be used, the same size being continued along each side of the web.

In cases where the stanchion section is less than that of the beam, it is only necessary to develop the strength of the stanchion in the connection. The construction will be generally as in Fig. 31, modified only to suit the conditions. Assume for calculation purposes the stanchion is of 8″ × 5″ joist section.

Section modulus of 8″ × 5″ R.S.J. = 22·42 ins.^{3}
Moment of resistance at 10 tons per sq. in. = 22·42 × 10
= 224·2 tons ins.

The maximum bending moment that can thus be transferred is 224·2 tons ins.

$$\text{Load transferred by flange welds} = \frac{224{\cdot}2}{11{\cdot}5} = 19{\cdot}5 \text{ tons.}$$

$$\text{Load per lineal inch of weld} = \frac{19{\cdot}5}{10} = 1{\cdot}95 \text{ tons.}$$

$\frac{7}{16}''$ fillets will take 2·0 tons per lineal inch and will be used.

As in Fig. 31, the stanchion web needs stiffening (if necessary) to resist shear, buckling, or bearing, and stiffeners will also be needed to the stanchion flanges capable of taking the load of 19·5 tons, the procedure being as already described.

Plate Girder Design.

Preliminary calculations for web and flange areas may be based on the assumption that the web takes all the shear and the flanges take the bending moment.

In Fig. 33—Let A_F = Flange area.
F_F = Flange force.
S_V = Vertical shear at any point.
S_H = Horizontal shear at same point.
f = Allowable flange stress.
M = Bending moment at any point.
M.R. = Moment of resistance at same point.

$$\begin{aligned} \text{Then } F_F &= A_F \times f \\ \text{M.R.} &= F_F \times d = A_F \times f \times d = M. \\ A_F &= \frac{F_F}{f} = \frac{M}{f \times d} \end{aligned}$$

Preliminary calculations for flange area may be made on this basis to find a suitable section quickly, and the result may be checked by the basic beam formula $f = \frac{M \times y}{I}$

The welds connecting the flanges to the web take the horizontal shear S_H, which may be calculated from the standard formula

$$S_H = \frac{S_V \times G}{I} = \frac{S_V \times A_F \times d/2}{I}$$

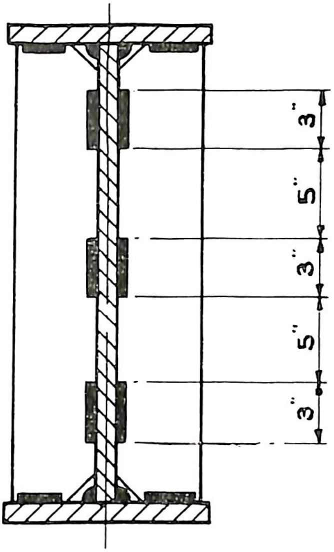
This formula gives the shear per lineal inch run of flange at any section, and the fillet size is fixed to suit, bearing in mind that there are two lines of fillet to each flange, so that each line takes half of the above horizontal shear per inch.

Fillets to the compression flange should always be made continuous, but those to the tension flange may be intermittent if the load permits, and providing there is no possibility of corrosion between the web and the flange.

In the case of plate girders taking heavy loads the flange plates will require to be thicker at the centre than at the ends, but unless the centre plates need to be very thick (say over 2″) it is not necessary to pile plates one on top of another as in riveted work. The centre portion of the flange can be made in one thickness (if not too heavy for handling) and the joint between this plate and the end plates can be made as shewn in Fig. 10, where the weld develops the strength of the thinner plate only. If with heavier loads it becomes necessary to add an additional plate in the centre, the outer plate is made narrower than the inner plate to allow for straightforward downhand welding, and the longitudinal welds connecting this plate to the lower plate will take the horizontal shear in the plane between the plates, the shear being calculated from the above formula using the area of the outer plate as the value of A_F.

Plate Girder Stiffeners.

Loads in the stiffeners are calculated in exactly the same manner as for riveted girders, and the welds connecting the stiffeners to the web must be sufficient to transmit the load in the stiffener. For welded work stiffeners usually consist of flats, and the welds may be intermittent if the stiffener load permits. As the stiffeners act as struts it is undesirable to have too large a gap between intermittent welds, 6″ being a fairly usual maximum. The inner



TYPICAL ARRANGEMENT OF STIFFENERS.

Fig. 34.

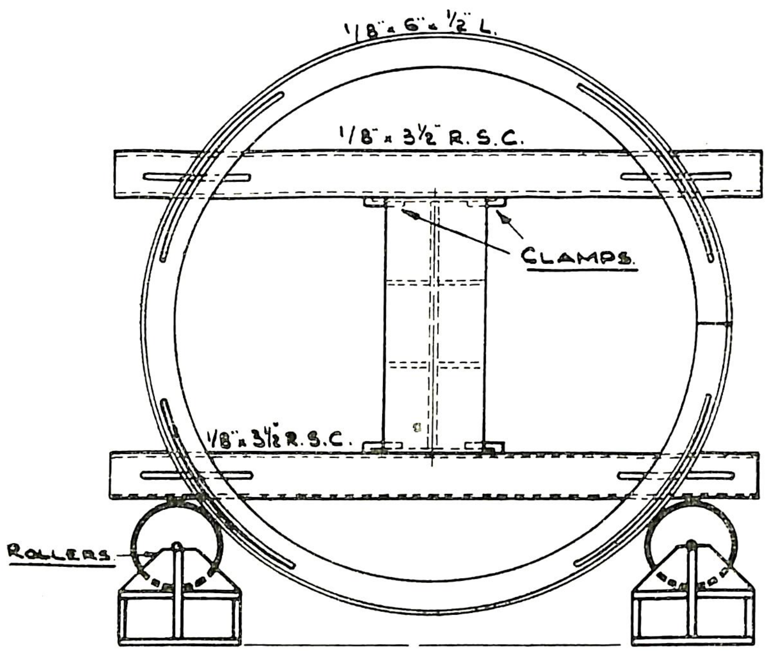
corners of the stiffeners should be bevel cut to clear the welds between the flange and the web. Fig. 34 shews a typical arrangement of plate girder stiffeners.

Generally it will be clear that so far as plate girders are concerned welded construction shews considerable economy in material compared with riveted work. In addition the construction is simpler, angle sections being completely eliminated, and providing adequate welding facilities are available the operations are comparatively straightforward, the jigs being so constructed that the girders can be rotated to enable all welds to be laid in a downhand position. The economy is particularly marked where a large number of girders are required of similar or generally similar construction, as the cost of jigs is then spread over a greater weight of work. For small quantities of girders, where special jigs have to be made for only one or two girders, the cost of jigs may cancel other economies. Against this, it is not particularly difficult to design adjustable jigs to serve girders of slightly different sizes. In any case the jig designer should always bear in mind that jigs for one job may be required at a later date to serve a different size of girder.

Procedure for the Welding of Plate Girders.

It should be regarded as a fixed rule in welding practice that as much welding as possible should be done in a downhand position. This is obviously the natural position which enables the operator to work in reasonable comfort and to produce good work. By comparison vertical and overhead welding are much more difficult and less likely to give a first-class job, and should never be attempted when downhand working can be arranged. On this basis the sequence of operations for welding a plate girder will be as follows :

1st Operation.—Lay one flange on the welding table or stallage, and assemble and weld all the web stiffeners to this flange.



JIG FABRICATED BY WELDING.

3 JIGS REQUIRED.
1 AT EACH END OF GIRDER.
1 CENTRAL.

DETAIL OF JIG FOR PLATE GIRDERS.

Fig. 35.

2nd Operation.—Lower the web plate into position between the stiffeners, and weld the web to the flange.

3rd Operation.—Turn the girder over on to its side to bring the web into a horizontal position. Assemble the second flange and clamp it to the stiffeners. Weld the near side of the web to the second flange and to the stiffeners.

4th Operation.—Turn the girder over with the other face of the web uppermost, and complete the welding.

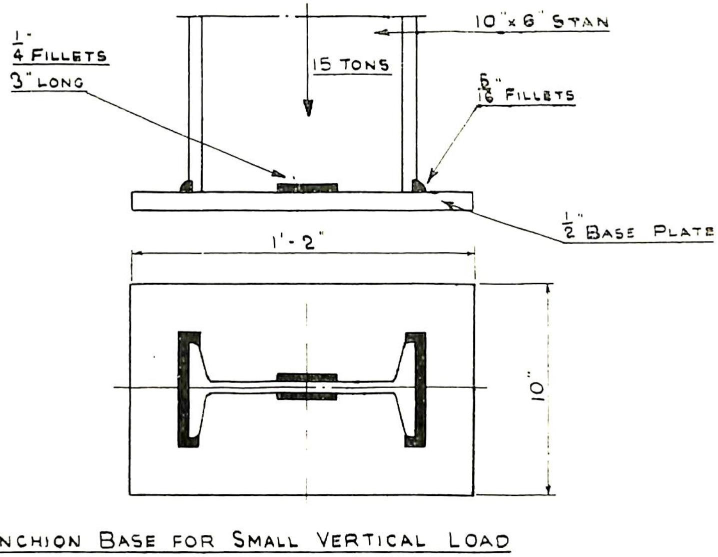
The above procedure may be adopted as a general guide for welded work of all kinds, and will obviate a number of difficulties met with in welding practice, giving the operator every opportunity to produce good work.

Jigs for Welding Plate Girders.

A simple but effective type of jig is shewn in Fig. 35, consisting of an angle ring resting on rollers, with two channel cross pieces for connecting to the girder flanges. Jigs of this type can be rotated to bring the girder into any desired position, while the arrangement of slots in the ring and the channels enables the jig to be used for girders of varying depth. With jigs constructed on this principle the cost per girder is very small.

Stanchion Bases—Subject to Vertical Load only.

The procedure is generally similar to that for beam connections of the rigid type, although when loads are small it is not necessary



STANCHION BASE FOR SMALL VERTICAL LOAD

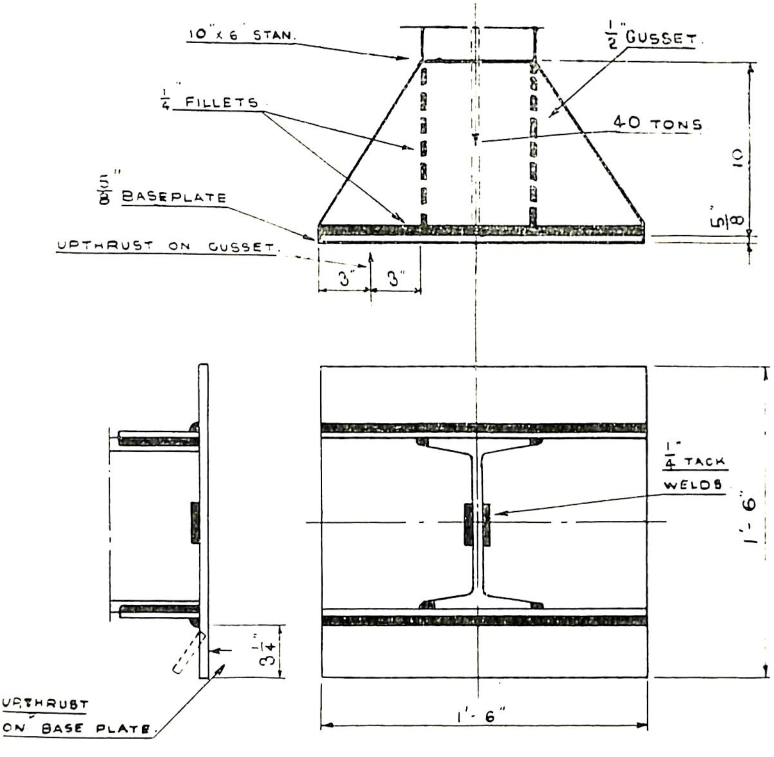
Fig. 36.

to obtain absolute rigidity. Fig. 36 shews a typical base for a stanchion supporting a small load, with all the load assumed to be transferred through the flange welds. Welds are returned round the profile of the flange to give a good finish and to ensure that the full width of 6″ is effective.

$$\text{Load per inch of flange weld} = \frac{15}{2 \times 6} = 1{\cdot}25 \text{ tons.}$$

$\frac{5}{16}$″ fillets will take 1·44 tons per inch and will be used. Short tack welds are included (without calculation) to the web for positioning purposes. The base plate may be designed as in the next example.

Fig. 37 shews a base supporting a heavier load. It will be assumed for safety that all the load is transmitted through the flange gussets, although a small proportion will almost certainly



STANCHION BASE FOR LARGER VERTICAL LOAD.

Fig. 37.

be transferred through the stanchion web. For the welds connecting the shaft to the gussets try $\frac{1}{4}''$ fillets at 1·15 tons per lineal inch.

$$\text{Effective length required in each fillet} = \frac{40}{4 \times 1{\cdot}15} = 8{\cdot}7''.$$

To cover for end craters and to give a commercial size for the gussets use 10″ deep flats.

For the welds connecting the gussets to the base plate try $\frac{1}{4}''$ fillets at 1·15 tons per lineal inch.

$$\text{Total effective length required} = \frac{40}{1{\cdot}15} = 34{\cdot}8''.$$

36″ total length is given and will be satisfactory. The gussets require sufficient area to resist the upthrust on the cantilevered portion of the base outside the shaft. Upthrust on 2 gussets on

$$6'' \text{ cantilever} = \frac{40 \times 6}{18} = 13{\cdot}33 \text{ tons.}$$

Checking for shear.

$$\text{Area of 2 gussets} = 2 \times 10 \times \tfrac{1}{2} = 10 \text{ sq. ins.}$$

$$\text{Shear stress} = \frac{13{\cdot}33}{10} = 1{\cdot}33 \text{ tons per sq. in.}$$

This is satisfactory.

Checking for bending.

$$\begin{aligned}\text{Moment of upthrust about edge of joist flange} &= 13{\cdot}33 \times 3\\ &= 40 \text{ tons. ins.}\end{aligned}$$

$$\begin{aligned}\text{Section modulus of 2 gussets} = 2\,\frac{BD^2}{6} &= \frac{2 \times \frac{1}{2} \times 10^2}{6}\\ &= 16{\cdot}67 \text{ ins.}^3\end{aligned}$$

$$\text{Bending stress} = \frac{M}{Z} = \frac{40}{16{\cdot}67} = \pm 2{\cdot}4 \text{ tons per sq. in.}$$

This stress should always be kept low as the upper half of the gusset is in compression, and plates have very little strength in compression. Strictly speaking, all stanchion base gussets should be stiffened along the edge to resist compression, but on small bases this is rarely done as the stress is usually small. The base plate thickness must be checked to resist upthrust on the projecting portion outside the gusset. The projection is taken as $3\frac{1}{4}''$, and the load on this portion of the base is

$$\frac{40 \times 3\frac{1}{4}}{18} = 7{\cdot}2 \text{ tons}$$

Moment of upthrust $= 7{\cdot}2 \times \frac{3\frac{1}{4}}{2} = 11{\cdot}7$ tons ins.

Section modulus of base plate $= \frac{BD^2}{6} = \frac{18 \times \frac{5}{8}^2}{6} = 1{\cdot}17$ ins.^{3}

Bending stress $= \frac{M}{Z} = \frac{11{\cdot}7}{1{\cdot}17} = \pm\ 10$ tons per sq. in.

This is satisfactory, especially as the loading conditions used are somewhat severe.

Shear stress in base plate $= \frac{\text{Upthrust}}{\text{Area}} = \frac{7{\cdot}2}{18 \times \frac{5}{8}}$

$= 0{\cdot}64$ tons/sq. in.

Stanchion Base Subject to Bending Moment.

Fig. 38 shews a typical welded steel base. The stanchion shaft is built up of two 12″ × ¾″ flange plates and one 18″ × ⅜″ web plate. A section of this type is equivalent to a broad flange beam and is very economical for stanchions subject to bending moment as the bulk of the area is in the flanges where it is most needed. A riveted section of equal strength would be composed of a rolled steel joist with flange plates, so that the welded construction saves all workmanship in drilling and riveting the flanges, the comparative labour required being the laying of four runs of weld connecting the web to the flanges. It is not necessary to continue the 12″ × ¾″ flange plates to the top of the stanchion; they can be reduced either in width or in thickness at a convenient point to suit the reduced bending moment, the joint being made with a butt weld.

The vertical axial load on the base is 25 tons and the bending moment due to wind is 800 tons-inches. The allowable pressure on the foundation is 30 tons per square foot.

Area of base plate $= 3{\cdot}5 \times 1{\cdot}67 = 5{\cdot}83$ sq. ft.

Direct pressure on foundation due to vertical load

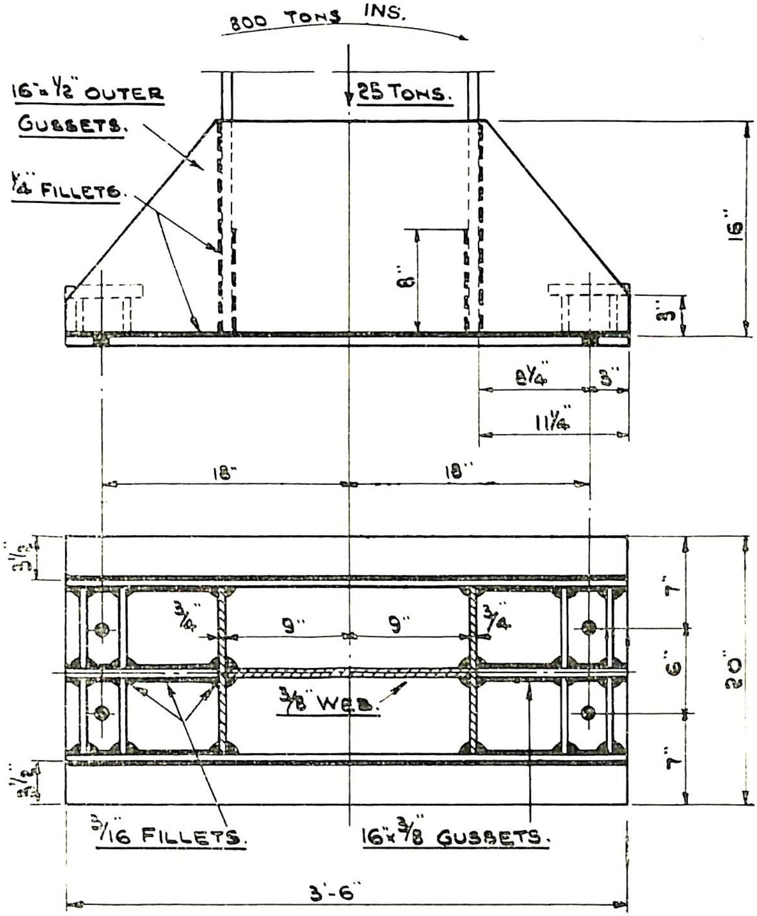
$= \frac{25}{5{\cdot}83} = 4{\cdot}3$ tons per sq. ft.

Section modulus of base plate area $= \frac{BD^2}{6} = \frac{1{\cdot}67 \times 3{\cdot}5^2}{6}$

$= 3{\cdot}4$ feet^3.

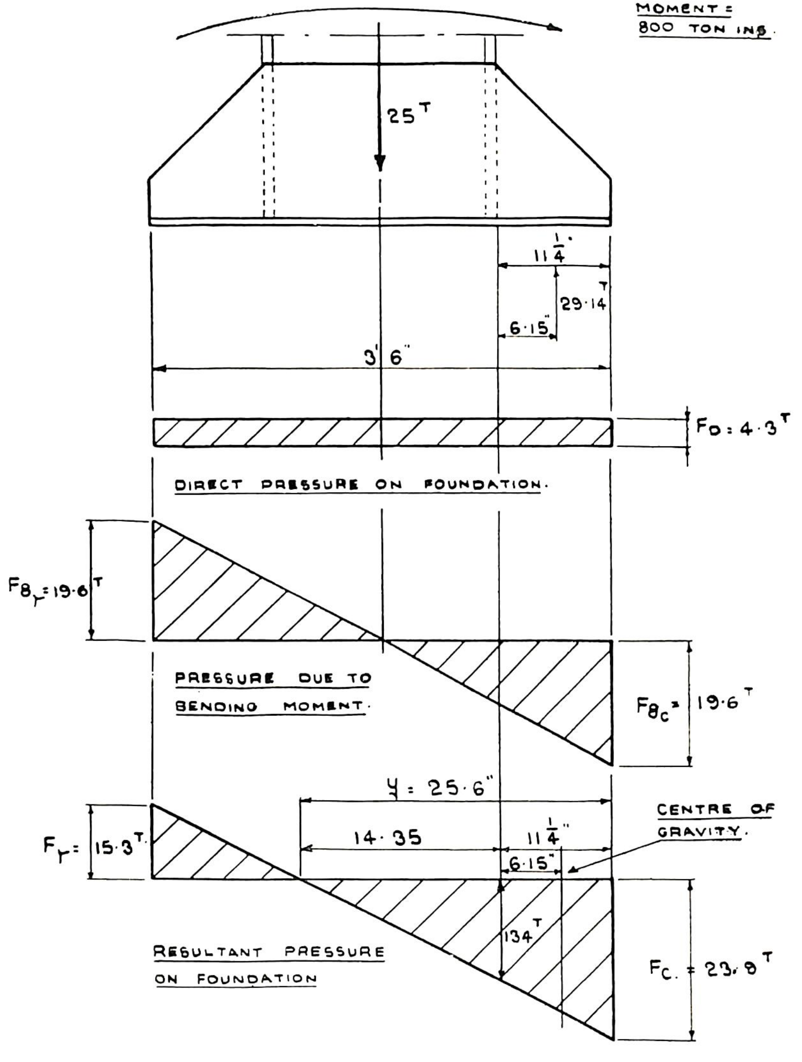
Pressure due to bending moment $= \frac{M}{Z} = \frac{800}{12 \times 3{\cdot}4}$

$= 19{\cdot}6$ tons per sq. ft.



DETAIL OF STANCHION BASE SUBJECT TO AXIAL LOAD AND BENDING MOMENT.

Fig. 38.



PRESSURE DIAGRAMS FOR STANCHION BASE

Fig. 39.

Maximum pressure on base $= 19{\cdot}6 + 4{\cdot}3 = 23{\cdot}9$ tons per sq. ft.
Tension on base $= 19{\cdot}6 - 4{\cdot}3 = 15{\cdot}3$ tons per sq. ft.

These pressures are set down in Fig. 39.

$$\text{Dimension } y = \frac{42 \times 23{\cdot}9}{39{\cdot}2} = 25{\cdot}6''$$

Pressure per sq. ft. at face of stanchion flange

$$= \frac{23{\cdot}9 \times 14{\cdot}35}{25{\cdot}6} = 13{\cdot}4 \text{ tons per sq. ft.}$$

Total upthrust on base outside stanchion face

$$= \left\{ 13{\cdot}4 + \left(\frac{23{\cdot}9 - 13{\cdot}4}{2}\right) \right\} \times \frac{11\frac{1}{4}}{12} \times 1{\cdot}67 = 29{\cdot}14 \text{ tons}$$

Acting at a distance from the stanchion face of

$$\frac{(13{\cdot}4 \times 5\tfrac{5}{8}) + (5{\cdot}25 \times 7\tfrac{1}{2})}{13{\cdot}4 + 5{\cdot}25} = 6{\cdot}15''$$

Moment of upthrust about stanchion face

$$= 29{\cdot}14 \times 6{\cdot}15 = 179{\cdot}21 \text{ tons ins.}$$

This moment must be resisted by the gussets, and is shared between the outer and inner gussets in proportion to the width of base each gusset is assumed to stiffen, that is, 7″ for the outer gusset and 6″ for the inner gusset (see Fig. 38).

Proportion of moment taken by each outer gusset

$$= \frac{179{\cdot}21 \times 7}{20} = 62{\cdot}72 \text{ tons ins.}$$

Proportion of moment taken by inner gusset

$$= \frac{179{\cdot}21 \times 6}{20} = 53{\cdot}96 \text{ tons ins.}$$

The outer gussets are 16″ × ½″ and the inner gussets 16″ × ⅜″.

$$\text{Section modulus of outer gusset} = \frac{BD^2}{6} = \frac{\frac{1}{2} \times 16^2}{6}$$

$$= 21{\cdot}33 \text{ ins.}^3$$

$$\text{Section modulus of inner gusset} = \frac{\frac{3}{8} \times 16^2}{6} = 16{\cdot}0 \text{ ins.}^3$$

$$\text{Bending stress in outer gusset} = \frac{62{\cdot}72}{21{\cdot}33}$$

$$= \pm\ 2{\cdot}94 \text{ tons/sq. in.}$$

Bending stress in inner gusset $= \dfrac{53{\cdot}96}{16}$

$= \pm\ 3{\cdot}37$ tons/sq. in.

These stresses are satisfactory, but are quite high enough as the upper half of the gussets are in compression and are unstiffened.

The gussets must also resist the upthrust in shear.

Area of outer gusset $= 16 \times \frac{1}{2} = 8$ sq. ins.

Shear stress $= \dfrac{7/20 \times 29{\cdot}14}{8} = 1{\cdot}28$ tons per sq. in.

Area of inner gusset $= 16 \times \frac{3}{8} = 6$ sq. ins.

Shear stress $= \dfrac{6/20 \times 29{\cdot}14}{6} = 1{\cdot}46$ tons per sq. in.

These stresses are satisfactory.

The welds connecting the gussets to the stanchion shaft must also resist the bending moment and the vertical load.

Force in welds due to bending moment

$$= \frac{\text{moment}}{\text{flange centres}} = \frac{800}{18{\cdot}75} = 42{\cdot}7 \text{ tons}$$

Force in welds due to vertical load $= \dfrac{25}{2} = 12{\cdot}5$ tons

Total force in welds $= 42{\cdot}7 + 12{\cdot}5 = 55{\cdot}2$ tons.

Proportion of force taken by outer gusset

$$= 55{\cdot}2 \times \frac{7}{20} = 19{\cdot}3 \text{ tons.}$$

Proportion of force taken by inner gusset

$$= 55{\cdot}2 \times \frac{6}{20} = 16{\cdot}6 \text{ tons.}$$

Using $\frac{1}{4}''$ fillets (as minimum) at $1{\cdot}15$ tons per lineal inch.

Length of weld required to outer gusset $= \dfrac{19{\cdot}3}{1{\cdot}15} = 16{\cdot}8''$.

Length of weld required to inner gusset $= \dfrac{16{\cdot}6}{1{\cdot}15} = 14{\cdot}4''$.

For the outer gussets the welds will be made the full depth of 16″ for the fillets outside the shaft and about 8″ for the inside fillets, this being approximately as far as the welds can be laid efficiently for such a construction. Welds to the inner gussets need slightly less strength for this condition, but also have to resist the uplift from the holding down bolts and will be checked along with the analysis for these bolts.

The base plate must resist the upthrust in the projecting portion of the plate outside the gussets. The maximum upthrust (at the leeward edge of the base) is 23·9 tons per square foot, reducing to zero at 25·6″ from this edge. As the bulk of the pressure is due to wind, it is usually considered safe to design the base plate to an assumed upthrust of 75% of the maximum.

Assumed upthrust per sq. ft. for design = 75% of 23·9
= say 18 tons.

Upthrust per inch run on cantilever of $3\frac{1}{2}''$

$$= \frac{18}{144} \times 3\tfrac{1}{2} = 0{\cdot}4375 \text{ tons.}$$

Moment of upthrust $= 0{\cdot}4375 \times 1\frac{3}{4} = 0{\cdot}7656$ tons ins.

For a permissible stress of 10 tons per sq. in.

$$\text{Section modulus required in plate} = \frac{0{\cdot}7656}{10} = 0{\cdot}0766 \text{ ins.}^3$$

$$\text{Section modulus given per inch run} = \frac{t^2}{6}$$

$$\therefore\ t \text{ (thickness)} = \sqrt{0{\cdot}0766 \times 6} = 0{\cdot}68''.$$

Use $\frac{3}{4}''$ plate.

The holding down bolts must be designed to resist the uplift due to the overturning moment, on the assumption that rotation takes place about the leeward bolts. In this they are helped by the vertical load which gives a resisting moment.

Let T = tension per bolt.
D = bolt centres, parallel to wind direction.
W = axial load.

$$\text{Then } (2\,T \times D) + (W \times \tfrac{1}{2}\,D) = \text{B.M.}$$
$$(2\,T \times 36) + (25 \times 18) = 800$$
$$T = 4{\cdot}86 \text{ tons.}$$

The bolts are also subject to a lateral thrust of 5 tons, due to wind.

Shear per bolt = 5/4 = 1·25 tons.

Try $1\frac{1}{4}''$ diameter bolts.

Area of shank = 1·227 sq. ins.
Area at bottom of thread = 0·894 sq. in.

$$\text{Tensile stress } (f_t) = \frac{4{\cdot}86}{0{\cdot}894} = 5{\cdot}44 \text{ tons per sq. in.}$$

$$\text{Shear stress } (f_s) = \frac{1{\cdot}25}{1{\cdot}227} = 1{\cdot}02 \text{ tons per sq. in.}$$

By the Theory of Principal Stresses.

Maximum shear stress V $= \sqrt{\left(\frac{f_t}{2}\right)^2 + f_s^2}$

$= \sqrt{2{\cdot}72^2 + 1{\cdot}02^2} = 2{\cdot}91$ tons per sq. in.

Maximum tensile stress F $= \frac{f_t}{2} + \text{V} = 2{\cdot}72 + 2{\cdot}91$

$= 5{\cdot}63$ tons per sq. in.

These stresses are satisfactory for wind loads.

The bolts must have a length sufficient to give an adequate adhesion value in the concrete foundation. The adhesion value of steel in concrete is commonly taken at 100 lbs. per square inch.

Circumference of bolt shank = 3·927 ins.

Adhesion value per inch run = $3{\cdot}927 \times 100 = 392{\cdot}7$ lbs.

For uplift of 4·86 tons and ignoring effect of washer plate at bolt head.

Minimum length of bolt buried in foundation

$$= \frac{4{\cdot}86 \times 2240}{392{\cdot}7} = 27{\cdot}7''$$

By including washer plates this length can be slightly reduced, to give a total bolt length of say 30″ overall.

The welds to the inner gusset can now be checked against uplift on the bolts, the pull on the gusset being equal to the tension on one bolt.

Tension per bolt = 4·86 tons.

Moment of this tension about stanchion face

$= 4{\cdot}86 \times 8\frac{1}{4} = 40{\cdot}1$ tons ins.

On the basis of 1″ throat thickness, with 2 runs of weld, bending

stress $= \frac{\text{BM}}{\text{Z}} = \frac{40{\cdot}1 \times 6}{2 \times 16^2} = 0{\cdot}47$ ton per sq. in.

Shear stress $\frac{4{\cdot}86}{2 \times 16} = 0{\cdot}15$ ton per sq. in.

Resultant stress $= \sqrt{0{\cdot}47^2 + 0{\cdot}15^2} = 0{\cdot}49$ ton per sq. in.

$\frac{1}{8}''$ fillets at $6\frac{1}{2}$ tons per square inch have a value of 0·57 ton per inch and would be sufficient for strength, but with a $\frac{3}{8}''$ plate $\frac{3}{16}''$ fillets will be used as a minimum, and on examination will also be found to be ample for the force of 16·6 tons per gusset previously found.

The outer gusset will be connected to the base plate with a $\frac{1}{4}''$ fillet, and all that is required here by way of calculation is to ensure that the length of fillet is at least equal to that connecting the gusset to the stanchion shaft, although when the gusset has an even bearing on the base plate this is not essential, as part of the load is transferred by bearing.

Welds connecting the web of the stanchion shaft to the flange must take the shear, consisting of the wind load plus any crane surge effect which may operate. The shear per lineal inch is the total lateral force divided by the depth of the web, 18″, and is usually small in stanchions of this type. To avoid corrosion these welds are usually made continuous and a $\frac{3}{16}$″ fillet will be ample.

Stanchion Crane Cap subject to Bending Moment.

Fig. 40 shews the layout of a typical welded crane cap. It cannot be claimed that the construction is very satisfactory, and the roof leg would be much stiffer if it were carried down to the first batten on the crane legs, but as the construction shewn is commonly used where economy of material is a consideration, it will be analysed.

It is necessary to check the strength of welds A, B and C, and also of the 17″ × 4″ channels.

Welds A.—The roof leg is welded to the diaphragm plate on both sides, and also across the top and bottom.

For simplicity a 1″ throat thickness is used for calculation as in previous examples.

$$I_{xx} \text{ of side fillets } = 4\frac{BD^3}{12} = \frac{4 \times 1 \times 16^3}{12} = 1366 \text{ ins.}^4$$

$$I_{xx} \text{ of top and bottom fillets (approx.) } = 4\,ar^2$$

$$= 4 \times 6 \times 1 \times 8^2 = 1536 \text{ ins.}^4$$

$$\text{Total } I_{xx} = 1366 + 1536 = 2902 \text{ ins.}^4$$

$$Z = \frac{I}{y} = \frac{2902}{8} = 362{\cdot}75 \text{ ins.}^3$$

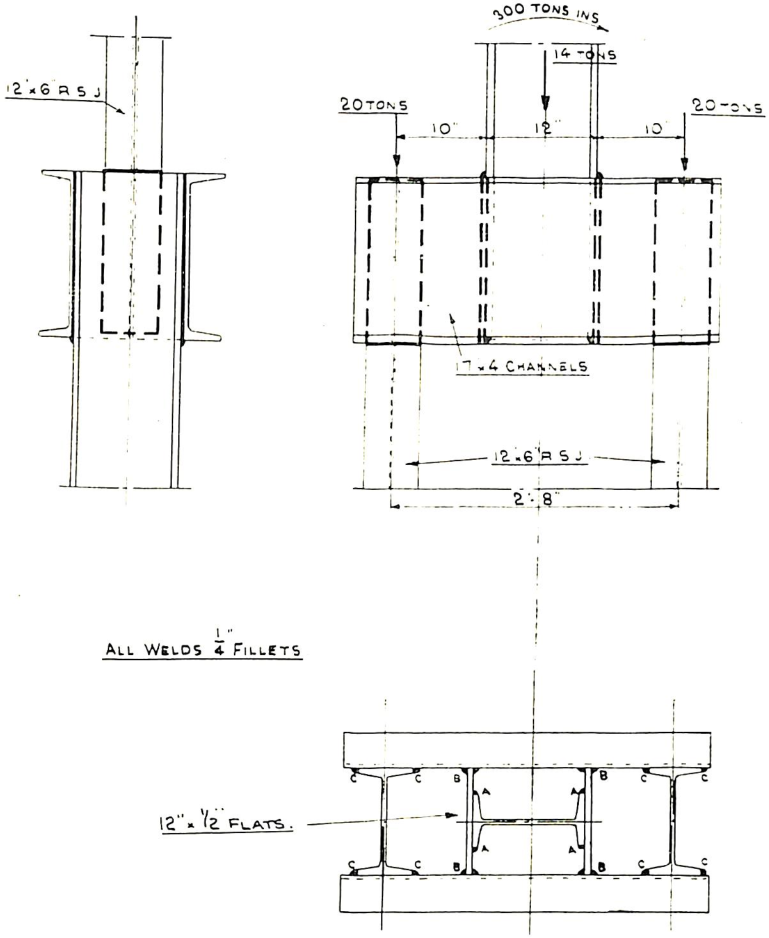
$$f_b \text{ (bending stress)} = \frac{M}{Z} = \frac{300}{362{\cdot}75} = 0{\cdot}83 \text{ ton per sq. in.}$$

$$\text{Total length of weld} = 4\,(16+6) = 88''.$$

$$f_s \text{ (shear stress)} = \frac{14}{88} = 0{\cdot}16 \text{ ton per sq. in.}$$

$$\text{Resultant stress} = \sqrt{0{\cdot}83^2 + 0{\cdot}16^2} = 0{\cdot}85 \text{ ton per sq. in.}$$

$\frac{3}{16}$″ fillets at $6\frac{1}{2}$ tons per square inch will take 0·86 ton per square inch, but as the construction generally is not altogether satisfactory and is liable to a certain amount of vibration it seems desirable to increase these welds to $\frac{1}{4}$″ fillets.



STANCHION CRANE CAP SUBJECT TO BENDING MOMENT.

Fig. 40.

Welds B.—Load per run of weld due to bending moment

$$= \frac{300}{12\cdot5 \times 8} = 3 \text{ tons.}$$

Load per run of weld due to vertical load $= 14/8 = 1\cdot75$ tons.
Total load per run $= 3 + 1\cdot75 = 4\cdot75$ tons.

$$\text{Load per inch run} = \frac{4\cdot75}{16} = 0\cdot3 \text{ ton.}$$

It is not desirable to use welds of less than $\frac{1}{4}''$ fillet especially as there will be some difficulty in laying the inside runs, and this size will be adopted.

Welds C.—These must take the bending moment of 300 tons-inches plus the roof load plus the crane reaction. The roof load will also induce a negative moment in these welds due to the fixed end condition, but this is small and providing the welds have a margin of strength can be neglected. The crane loads are rolling loads and the maximum reaction of 20 tons will not therefore be divided equally between the two flanges of the crane leg. It is difficult, if not impossible, to say with exactness what will be the maximum crane load entering each flange of the crane leg in turn, but a fairly usual and safe assumption is to allow for 75 per cent of the total reaction being taken by each flange in turn.

On the above basis, assumed load per run of weld

$$= \frac{300}{4 \times 32} + \frac{14}{8} + \frac{15}{2} = 11\cdot6 \text{ tons.}$$

$$\text{Load per inch run of fillet} = \frac{11\cdot6}{16} = 0\cdot73 \text{ ton.}$$

$\frac{1}{4}''$ fillets will be used again, and will give an ample margin of safety for the small fixed ended moment referred to above.

17″ × 4″ Channels.

These take the roof load of 14 tons plus the bending moment of 300 tons-inches. With four lines of weld at each reaction the channels may be assumed to be fixed-ended, and the positive moment may be calculated by the standard procedure for fixed ended beams.

Referring to Fig. 41 :

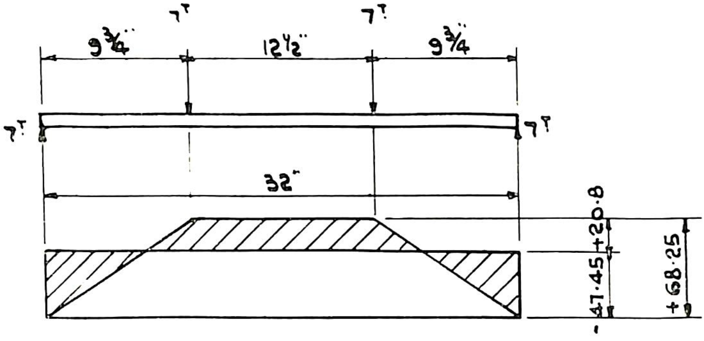
B.M. for free-ended condition $= 7 \times 9\cdot75 = 68\cdot25$ tons ins.

Area of free-ended B.M. diagram $= 68\cdot25\ (9\cdot75 + 12\cdot5)$

$= 1518\cdot56$ tons ins.^{2}

$\therefore$ Area of negative moment diagram $= 1518\cdot56$ tons ins.^{2}

$$\therefore \text{ Negative moment at ends} = \frac{1518\cdot56}{32} = 47\cdot45 \text{ tons ins.}$$



B.M. DIAGRAM FOR 17"×4" CHANNELS IN FIG 40.

Fig. 41.

$\therefore$ Positive moment at centre $= 68{\cdot}25 - 47{\cdot}45 = 20{\cdot}8$ tons ins.

Total positive moment $= 300 + 20{\cdot}8 = 320{\cdot}8$ tons ins.

$$f_b \text{ (bending stress)} = \frac{\text{B.M.}}{Z} = \frac{320{\cdot}8}{2 \times 61{\cdot}2}$$

$$= 2{\cdot}62 \text{ tons per sq. in.}$$

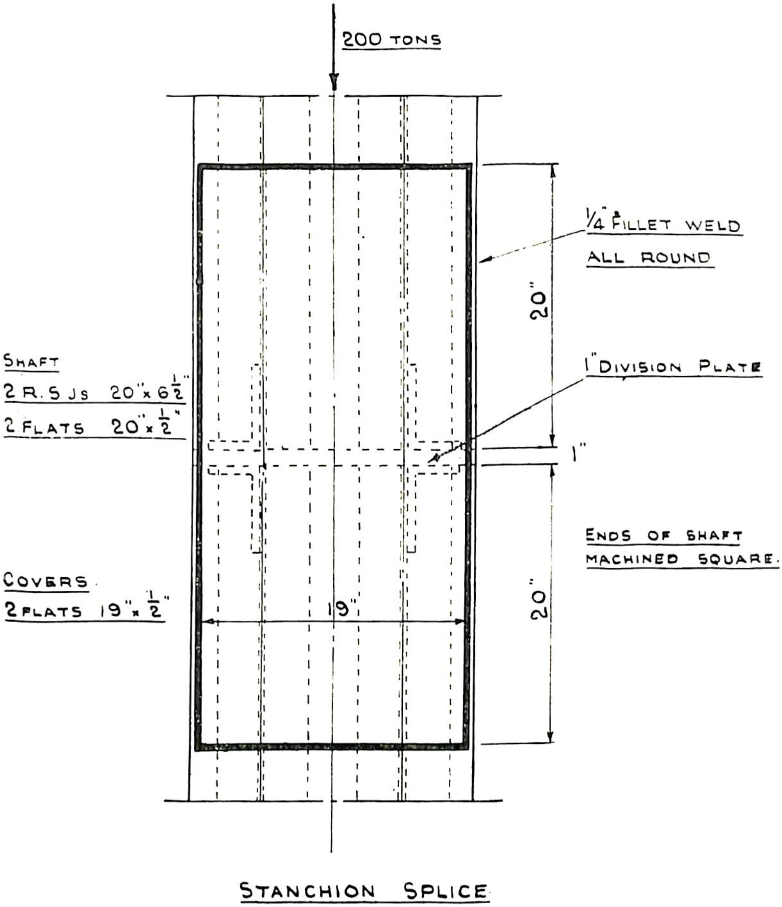
$$f_s \text{ (shear stress)} = \frac{14}{4 \times 17 \times 0{\cdot}48} = 0{\cdot}43 \text{ ton per sq. in.}$$

These stresses are satisfactory, and it will be seen that the important point about such a construction as that shewn in Fig. 40 is to provide adequate depth to give sufficient lengths of weld run and to ensure a reasonable degree of rigidity.

Stanchion Splices.

In multi-storey buildings splices are usually located immediately above a floor, and it is not therefore very difficult to weld the joint covers at site. Fig. 42 shows a typical welded splice for a column taking vertical load only. When the shaft ends are machined, specification requires that the covers shall be sufficient to hold the connected parts accurately in position, and the effective length of the splice from the plane of the joint is to be not less than the width of the spliced flange. It will be seen that Fig. 42 satisfies these requirements, and no calculations are required. It is necessary to run the weld all round the cover, to ensure a reasonably equal distribution in the cover of whatever load it may take.

It may be noted that with a total length of weld of 59″ on each side of the joint, the welds will take about two-thirds of the vertical



STANCHION SPLICE

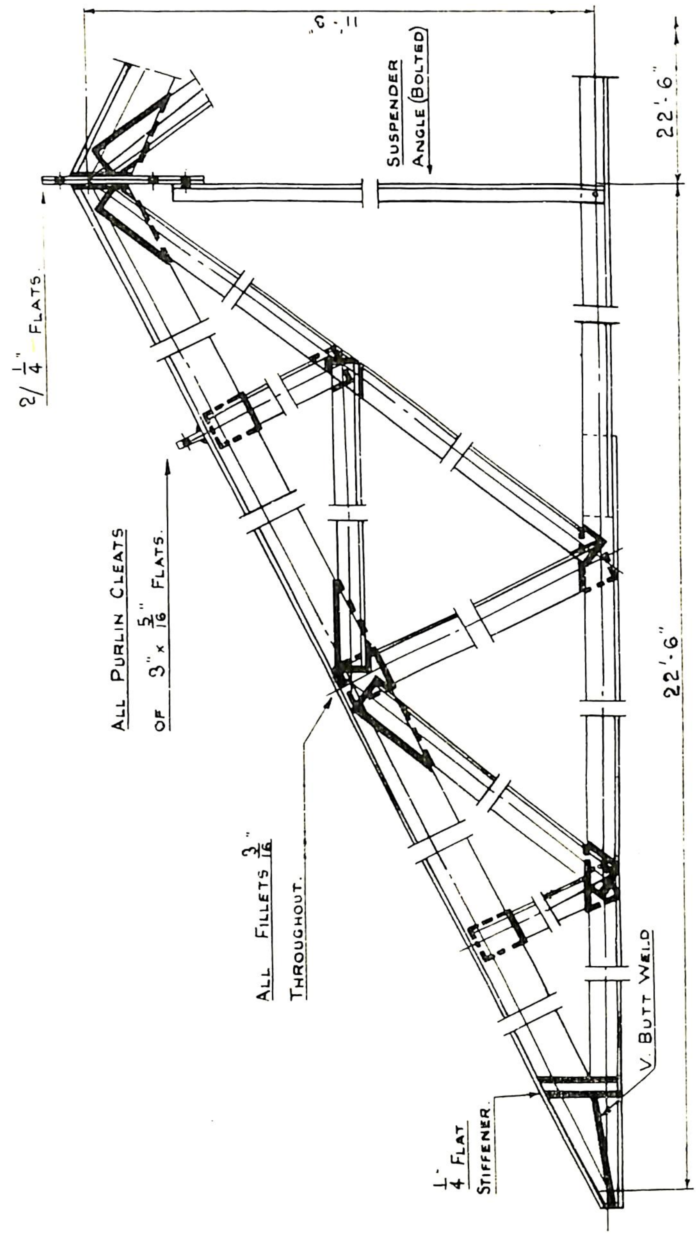
Fig. 42.

load which will be ample to cover the effects of any inaccuracy in bearing at the joint.

The cover is made slightly narrower than the flange plates of the shaft, to allow the welds to be laid as simple fillets on a flat surface.

Roof Truss Details.

Loads in roof truss members are mostly small, and generally there is no difficulty in arranging welds of adequate strength without the necessity of introducing gusset plates. Attention must therefore be given chiefly to arranging the details of the joints



Typical Details of Roof Truss.

Fig. 43.

to facilitate welding procedure. In Fig. 43 the vertical leg of the rafter and of the main tie are in the same plane, with a V-butt weld at the shoe. The outstanding legs of all the web members turn the same way as those of the rafter and main tie, but while the tension members connect to the near face of the chords the struts have their outstanding legs notched away at the ends to enable them to be welded to the far face of the chords. By this procedure it is fairly easy to arrange the weld runs and to fix up the sequence of operations.

For large trusses the disposition and length of welds is arranged in accordance with the procedure given for Figs. 17 or 18, whichever is applicable, but for smaller trusses the length of weld provided is usually in excess of theoretical requirements and the only consideration is arranging a convenient detail.

The centre portion of the main tie and the centre suspender angle are despatched loose and site bolted to eliminate all site welding. The detail at the apex is arranged for the same purpose and is quite satisfactory. The construction thus provides a typical example of the combination of shop welding and site bolting.

Elimination of gussets effects a substantial saving in material, while the absence of rivet holes in the tension members also enables smaller members to be used than would be the case with riveted work. The struts may also be designed as fixed-ended, which may give further economy. With all these savings it may be assumed that a welded truss without gussets will weigh about 15% less than the corresponding riveted truss.

The principal difficulty with welded trusses, and with all braced frames generally, is in clamping the members together in correct relative position, and this to some extent offsets economies in weight. Allowing for this difficulty, welded trusses will still tend to work out cheaper than riveted trusses, providing adequate facilities are available for carrying out the various operations efficiently.

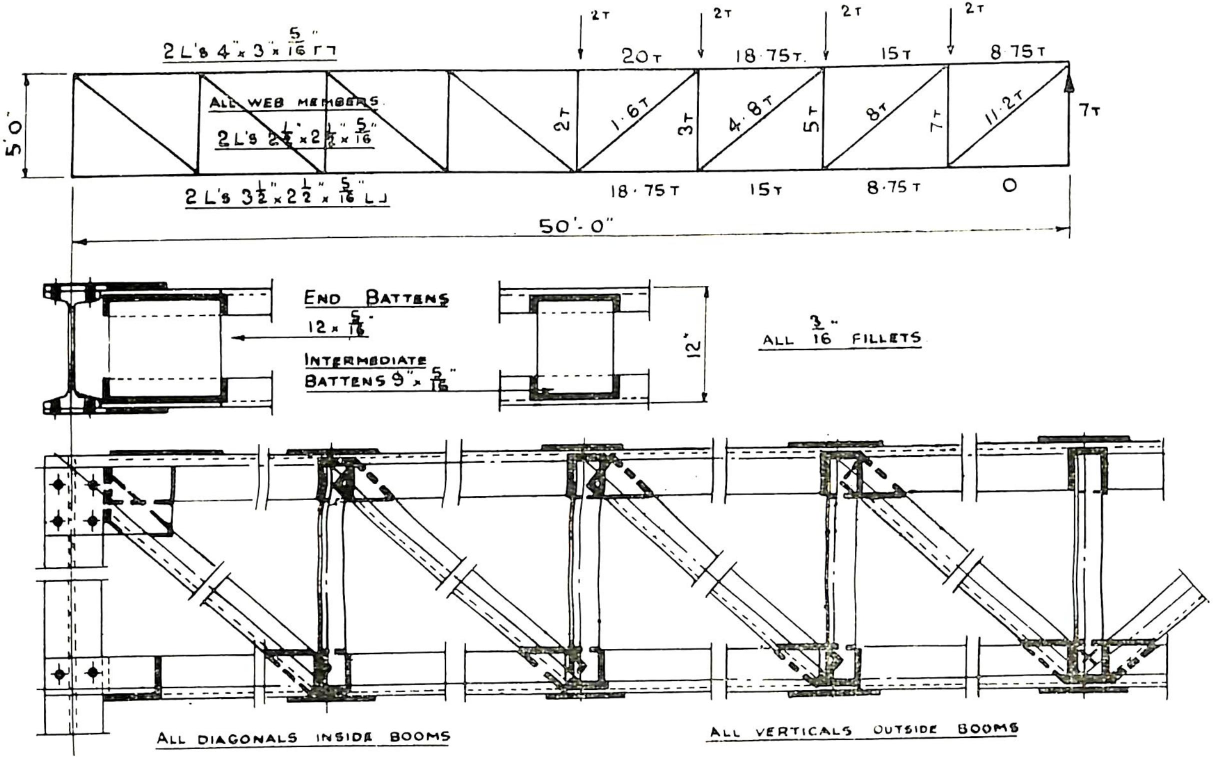
It will be noticed in Fig. 43 that all web members have welds on each face of the truss. This is important, as the sequence of operations is based on this principle, the procedure being as follows :

1st Operation.—For each half truss, clamp all members firmly in correct relative position, the truss lying flat on the assembly stallage.

2nd Operation.—Lay all welds on the upper face of the half truss, thus tacking all members in position.

3rd Operation.—Turn the half truss completely over, and complete by laying all welds on the other side.

Thus it will be seen that downhand welding is employed throughout, so that once the members are securely clamped in position no difficulty should be experienced.



TYPICAL DETAILS OF LATTICE GIRDER

Fig. 44.

With larger trusses where it may not be possible to get in sufficient weld metal to transfer the load in the web members without gussets, a convenient method is to butt-weld gussets to the vertical legs of the rafter and main tie at the joints in question, the gussets being the same thickness as the member to which they are welded, the effect being to widen the member at the joint.

Lattice Girder Details.

Fig. 44 shows a typical layout for a 50′ 0″ span lattice girder. The chords have been opened out partly to give lateral rigidity to the girder and also to allow the diagonals to be welded to the inside faces. The verticals have been located outside the chords to facilitate welding arrangements. If considered desirable to keep all members as nearly as possible in the same plane, the legs of the diagonals can be turned outwards and notched at the ends to enable these bars still to be welded to the inside faces of the chords, but as the diagonals are tension members this is not a vital point on small span girders, where both the sections and welds are usually of ample strength.

Weld lengths may be fixed up as explained for Figs. 17 or 18, and the weld arrangement at each joint follows the same general principle as for the roof truss. Each member has a near side and far side weld at both ends. Each half girder will be laid out, the members clamped together, and all the near side welds laid. Then the half-girder will be turned completely over and the welds laid to the other side. Finally the two half-girders are brought together and the batten plates welded on, all operations being by downhand welding.

Staircase Details.

Fig. 45 shews a typical layout of a welded staircase stringer and should be self-explanatory. It will be seen that compared with a riveted construction details are much simpler. The stair treads are supported by flats acting as lugs and the channel cutting at the top gives a more straightforward and "natural" construction than is possible for a riveted joint, which in this particular case is often complicated and costly.

Standardising of Welding Technique.

It is not possible to lay down hard and fast rules governing procedure and assessing electrode requirements, as most firms have their own individual standards, all of which will vary slightly according to circumstances and possibly due to personal preference.

A considerable variety of electrodes are in use, and in welded work more even than in riveted work plant layout has a considerable effect on performance, while of equal importance is the need

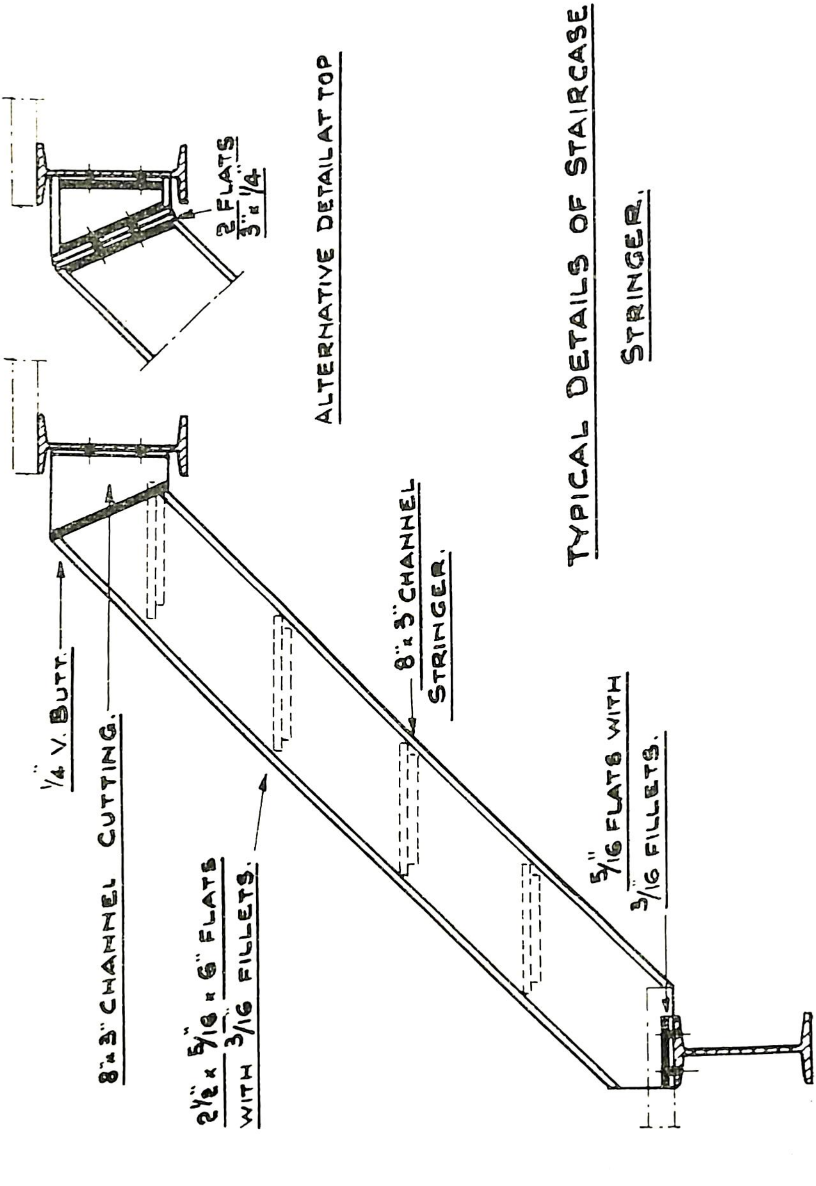

TYPICAL DETAILS OF STAIRCASE STRINGER.
ALTERNATIVE DETAIL AT TOP
2 FLATS 3" x 1/4"
1/4" V. BUTT.
8" x 3" CHANNEL CUTTING.
2½" x 5/16" x 6" FLATS WITH 3/16" FILLETS.
8" x 3" CHANNEL STRINGER.
5/16" FLATS WITH 3/16" FILLETS.

Fig. 45.

for making provision for the operator to work continuously in reasonable comfort without undue fatigue. All these points should be borne in mind in fixing up procedure and assessing electrode quantities. A certain amount of general guidance can be given and the following notes give an approximate basis for procedure and calculation, always bearing in mind that they need checking to suit individual circumstances.

It can be stated that it is economical to deposit any given size of weld with the largest electrode the work will stand. For down-hand welding when no special difficulty is likely, electrodes should be 10 or 12 gauge for plates under $\frac{3}{16}''$ thick, 8 gauge for plates $\frac{3}{16}''$ to $\frac{5}{16}''$ thick, 6 gauge for $\frac{5}{16}''$ to $\frac{7}{16}''$, and 4 gauge for $\frac{1}{2}''$ or over. Heavier electrodes $\frac{5}{16}''$ or $\frac{3}{8}''$ diameter are sometimes used for thicker plates.

For vertical, inclined, and overhead welding, these sizes are likely to vary slightly, and reference should be made to the shops before specifying rod gauges in these cases.

Electrodes are normally supplied in standard lengths of 18″, and the approximate lengths of run which should be obtainable from standard electrodes for fillet welds and for single V-butt welds are given in Tables 3 and 4. These values are for normal straightforward work and can only be approximately correct as they will vary according to the fit-up of the work and according to the skill of the welder. Allowance has been made in the tables for average wastage, although the actual amount of waste is somewhat indeterminate. With heavy electrodes it will be less than with light electrodes. It will also be less for long runs than for short runs. Inevitably it will also vary with individual operators. For 10-gauge electrodes it will average 15% to 20% with a further 5% for short runs. For 4-gauge electrodes it will be about 10% to 15% with 5% extra for short runs. In actual practice it is rare that the estimated total electrode requirements agrees with the number of electrodes used, though with so many conditioning factors this does not necessarily imply "faulty" estimating.

Values given in Tables 3 and 4 are based upon actual practice of average structural work, with welders of average skill working under reasonably satisfactory conditions.

Weld runs are normally specified as 10/12, 8/10, 6/9, etc., the first number indicating the gauge of the electrode and the second number the length of run in inches which may be obtained from a standard 18″ electrode.

Notation and Symbols for Welds.

Until recently most companies have built up their own system of notation and symbols, based upon experience, and providing these are generally understood by drawing office and works there

TABLE 3.

Approximate Lengths of Run obtainable from Standard 18″ Electrodes for Fillet Welds.

Fillet Size	Throat Thickness	Area Sq. ins.	Gauge of Electrode	Length of Run
$\frac{1}{8}''$	0·088″	0·012	12	10″
$\frac{1}{8}''$	0·088″	0·012	10	14″
$\frac{3}{16}''$	0·133″	0·023	10	8″
$\frac{3}{16}''$	0·133″	0·023	8	13″
$\frac{1}{4}''$	0·176″	0·039	8	8″
$\frac{1}{4}''$	0·176″	0·039	6	12″
$\frac{5}{16}''$	0·221″	0·058	6	8″
$\frac{5}{16}''$	0·221″	0·058	4	12″
$\frac{3}{8}''$	0·265″	0·081	4	8″

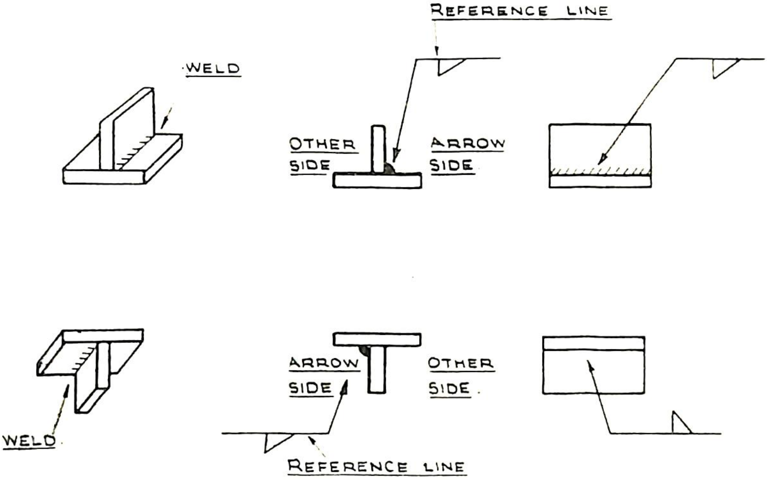
TABLE 4.

Approximate Lengths of Run obtainable from Standard 18″ Electrodes for Single V-Butt Welds.

Plate Thickness	Area Sq. ins.	Gauge of Electrode	Length of Run	Angle of Weld
$\frac{1}{8}''$	0·019	12	14″	70°
$\frac{1}{8}''$	0·019	10	18″	70°
$\frac{3}{16}''$	0·037	12	9″	70°
$\frac{3}{16}''$	0·037	8	14″	70°
$\frac{1}{4}''$	0·060	8	7″	70°
$\frac{1}{4}''$	0·060	6	10″	70°
$\frac{5}{16}''$	0·088	6	7″	70°
$\frac{5}{16}''$	0·088	4	9″	70°

Sealing Run.—One 10/12″ in each case in addition.

is nothing against this for strictly internal purposes. When drawings are made for outside distribution there is inevitably a certain amount of confusion caused by the different systems in use, and it is highly desirable that notations and symbols should be standardised. This has been done in B.S. 499, Section 7, 1952, "Scheme of Symbols for Welding," and it is strongly recommended that this standard should be generally adopted. It is not possible to reproduce here the complete recommendation of this standard and reference should be made to the specification for detailed procedure. Particular attention is drawn however to the following points :—



WELD SYMBOLS.

Fig. 46.

1. The foreword states that "the scheme departs from previous practice in two fundamental respects. The terms "near side" and "far side" have been discontinued in favour of the terms "arrow side" and "other side." This not only provides uniformity with the American scheme, but overcomes the difficulty encountered in some views of welds where the former terms were inapplicable. It has also been decided that the disposition of the weld symbol relative to the reference line should be based on the American system. This latter decision means that the symbol is placed below the reference line for welds on the "arrow side" and above the reference line for welds on the "other side." Two examples of the proposed procedure are given in Fig. 46, which serve to indicate the general principle, and a complete range of symbols

for all types of welds are given in the specification, to which reference should be made.

2. Clause 3 requires that the type of weld shall be indicated by the appropriate symbol. The vertical portions of the symbol for fillet, single-bevel butt, double-bevel butt, single-J butt, and double-J butt welds shall be on the left-hand side of the symbol, irrespective of the orientation of the weld metal. This is shewn in Fig. 46.

3. Clause 4 requires that in order to provide a complete indication of the position of a weld in a particular joint, relative to the parts being joined, the head of an arrow shall be used to indicate the reference line of the joint. The side nearer the arrow head shall be known as the "arrow side" and the remote side the "other side."

4. Clause 5 draws attention to the fact that the reference line, which is connected to the arrow at an angle, is an arbitrary datum line for determining the position of the weld symbol in order that the latter may indicate the location of the weld. This indication shall be provided as follows :—

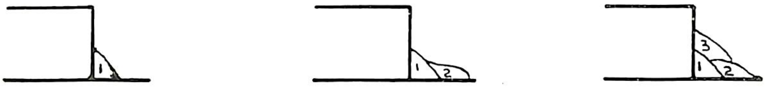
(*a*)	For welds to be made from the arrow side of a joint.	By inverting the symbol and suspending it from the reference line.
(*b*)	For welds to be made from the other side of a joint.	By basing the symbol upon the reference line.
(*c*)	For welds to be made on both sides of a joint.	By disposing the symbol about the reference line.

5. Clause 6 states that in the case of joints in which only one plate is prepared, the arrow shall point towards that plate, the direction of the edge preparation being indicated by the disposition of the symbol relative to the reference line.

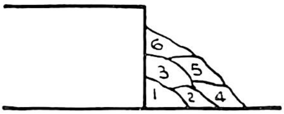
6. Instructions are given in the specification regarding the method of indicating weld sizes and weld lengths, both for continuous and intermittent welds, and also for distinguishing between shop and site welds.

Multiple Run Welds.

For weld sizes larger than $\frac{1}{4}''$ it is better practice and generally more economical to lay them in two or more runs. This is sometimes disputed so far as fillet welds are concerned, but there can be little doubt that this is so with butt welds. Generally speaking,



3 RUN WELD SEQUENCE & PROFILE OF INDIVIDUAL RUN.



6 RUN WELD.

MULTIPLE RUN WELDS.

Fig. 47.

two-run welds are undesirable as they necessitate weaving the second run to obtain the required profile, and weaving is always suspect from the point of view of strength. If more than a single run is necessary it is better to have three runs rather than two. The general build-up of multiple run welds is shown in Fig. 47. It will be seen that the profile of each run is so rounded as to prevent slag from the electrode coating percolating into the weld. This point is most important, requiring strict attention, and each run should be inspected and passed before the next run is deposited. If a neat final profile is required, a weaving run can be added, but this will give very little added strength to the weld and no account should be taken of it in calculations.

ACKNOWLEDGMENTS.

Extracts from B.S. 449, 1948, "The Use of Structural Steel in Buildings," B.S. 499, Section 7, 1952, "Scheme of Symbols for Welding," B.S. 538, 1940, "Metal Arc Welding in Mild Steel as Applied to General Building Construction," B.S. 1856, 1952, "General Requirements for the Metal Arc Welding of Mild Steel," are given by permission of the British Standards Institution, 24-28 Victoria Street, London, S.W.1, from whom official copies of these standards can be obtained, price 6/-, 6/-, 2/6 and 5/- respectively.

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23. 1" ,, ,, ,, ,,
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25. ⅞" ,, ,, ,, (30 ton yield).
26. 1" ,, ,, ,, (30 ton yield).
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